STRAINS IN IRONWORK.

A COURSE OF

EIGHT ELEMENTARY LECTURES

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PREFACE.

The following lectures were delivered during the Session 1882–3, in the Hall of the Society of Engineers, at the request of the Council of the Society, who arranged for several courses of lectures of a character calculated to prove of practical utility to the junior members of the profession, and of which the present formed the first course. They are now published with a view to their being used as an introduction to a more advanced course of lectures which is in contemplation.

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STRAINS IN IRONWORK.

LECTURE I.

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In the course of lectures commenced this evening, I shall place before you the subject of Strains in Ironwork in as simple and practical a manner as I am able, assuming that you know nothing of mathematics beyond the first elements of arithmetic, and I shall endeavour, without going higher than the rules of simple proportion, to make clear to you all you require to learn in order to determine the amount of stress in the different parts of a structure, from a simple bolt or rivet to the complete framing of some of the gigantic works in iron now called for by modern engineering. The importance of this study cannot be overrated, whether we consider the Civil Engineer and the Architect who give the first conception of the structure, the Draughtsman who works out the detail of the design, or the Mechanical Engineer and Con-

tractor who carry it into practice; everyone who has to do with either the design or the execution must have a more or less complete knowledge of the stresses and strains, in order that his portion of the work may be successfully carried out. The want of this knowledge is often apparent in the works of Architects: the use of iron is comparatively new to them, although now indispensable, and they are too often satisfied to leave the contractor to ascertain the necessary amount of material: a practice which, there is little doubt, has sometimes led to disastrous results; but as the contractor is paid by weight, the error is generally on the safe side.

A knowledge of the physical properties of iron, and the mechanical details of its formation into members and structures, form of course a large part of the stock-in-trade in designing, but the "calculation of strains" as it is called must be looked upon as certainly the most important item. This tells you what amount of stress must be provided against in the various parts, but it does not tell how to proportion the shape of the parts. It clothes a skeleton of the structure with figures representing the forces to be resisted, and undertakes all the theoretical portion of the work, preparing it for the practical designer, who, in place of the figures, draws in the breadth and thickness of the parts.

It will be necessary briefly to call your attention to a few facts connected with the material, in order that you may appreciate the different forms we shall investigate.

Iron is known under three principal varieties: wrought iron, steel, and cast iron; and, strange as it may seem, the only essential difference between these three varieties is the amount of charcoal or carbon combined with the iron. The purest is Wrought Iron, it contains not more than ¼ lb. of carbon in 100 lbs. of iron. It is *malleable*, can be forged or hammered into various shapes at a red heat. It is *ductile*, can be rolled into plates and bars, drawn into rods of different sizes and even into fine wire. It is *weldable*, pieces can be

joined together at a white heat by hammering them, making one solid piece equally strong throughout. It is *soft*, considered as a metal, and *fibrous* or stringy with a grain somewhat after the nature of wood, running in the direction of its length. It is *tough* and *elastic*. It rapidly *oxidises* or rusts in damp climates, if unprotected by paint or otherwise.

STEEL is wrought iron with a little more carbon in it, say 1 lb. in 100 lbs. of iron, but this slight addition of carbon makes a vast difference in many of its properties. There are other alleged constituents in steel, but the evidence is incomplete, and need not trouble us at present. The proportion of carbon also varies, steel beginning where wrought iron leaves off; in fact, the line of demarcation is so slight, that there are often samples of metal which one man will call iron and another steel, even after an elaborate chemical analysis. Steel is *malleable* and *ductile*, but requires more care in working than wrought iron, and a lower temperature. It is *weldable* if the carbon is not present in too great a proportion, but when the carbon is in excess the steel can be melted and run into moulds like cast iron. The special and significant property of steel is its ability to *temper*, as it is called, or take various degrees of hardness, when made red hot and plunged into water, which neither wrought nor cast iron will do. Its texture varies from *fibrous* to *crystalline* or granular. It is much *tougher* than wrought iron, and more *elastic*, and generally speaking, it would be described as *stronger*.

CAST IRON is wrought iron with a large admixture of carbon, say 2 to 5 lbs. in 100 lbs. of iron. It is *brittle*, *inflexible*, and *crystalline*, neither malleable nor ductile, cannot be welded, but it may be *melted* at a high temperature and poured into *moulds* or impressions in sand, forming *castings* of the shape required.

As you might anticipate from this description, one form of iron may be changed into another by the addition or sub-

traction of carbon; and this is actually done, but the method and object do not concern us just now. There are a few intermediate and allied stages which may be noticed; for instance, wrought iron may have the surface converted into steel, called *case-hardening*, in order to give a rubbing surface of steel without incurring the expense of steel for the whole piece. Cast iron may have the surface hardened or *chilled* in the process of casting; this is done where a very hard surface is required to resist the wear and tear, the piece not being subject to much strain. Cast iron is sometimes partially converted into wrought iron, when it is required to be tougher than the ordinary cast iron, but cheaper than forged wrought iron: this is called *malleable cast iron.* Malleable iron and rolled iron are only other names for ordinary wrought iron. *Mild steel* is only one remove from wrought iron, it has all the properties of steel but very slightly or mildly. Cast iron is sometimes spoken of only as *metal;* at other times the term metal is used for brass and gun metal; and again, it is used in speaking of the dimensions in the same way as the word scantling. The chief and distinctive terms to be used are—wrought iron, steel, and cast iron.

There are two principal varieties of fastenings adopted in connection with ironwork, viz., bolts and rivets. Bolts are pieces of round iron with a solid head at one end and a loose nut screwed on at the other. Rivets are also pieces of round iron with a solid head at one end, but they are plain at the other end. When used the rivet is made red hot, put through the pieces to be joined, and held firmly, while another head is formed by hammering the point down. On account of the hammering required, riveting is not suitable for cast-iron structures, as the metal is too brittle to withstand the shocks; it is fortunately not much wanted, however, as castings may be made very large and require few joints. Wrought iron is mostly riveted, as it is a cheap and permanent mode of connecting the parts, but no rivets are used larger than one inch

diameter in ordinary work; when such would be required, bolts are resorted to. Instead of having two or three bolts or rivets to connect a joint, the connection is sometimes made by pins or cotters, a single pin being often three or four inches diameter. The pin when used alone allows of some movement, or "hinge-like" play in the joint, which is in some cases desirable.

The strength of any piece subject to direct strain depends upon the size or sectional area of its smallest part, and in a whole structure the strength is similarly measured by that of its weakest part. Roughly speaking, we may say that twice the size gives twice the strength, but there are important modifications of the rule, which we shall observe in due course. The art of designing economically consists in providing the maximum of strength with the minimum of material, and there would be no scope in this respect if the size alone determined the strength.

There are three terms used in such discussions as the present, which I think it necessary to direct your careful attention to. They are—*load*, *strain*, and *stress*. By a LOAD on any member of a structure is meant the aggregate of all the external forces acting upon it, including the weight of the member itself, and of other parts supported by it. By a STRAIN is meant the change of form produced in a piece by the action of a load; and by a STRESS is meant the resistance set up in the material, by its molecular forces opposing the action of a *load* in producing a *strain*. Stress and strength may be looked upon as synonymous terms, although they are not used exactly alike. Thus the *strength* of a piece in a given position may be such, that a *load* of so many *lbs.* produces a *stress* of so-many *lbs. per square inch*, the result being a *strain*, or change of form of a certain amount, whether temporary or permanent, and, when large enough, appearing as stretching, shortening, bending, crumpling, or twisting. The term strain is commonly used instead of stress, and is usually meant as

such when we speak of the calculation of strains; so that although we may use this term as an ordinary office phrase, we must bear in mind the distinction when we make a systematic study of the subject.

The various kinds of strain which can come upon any member of a structure are:

TENSION: stretching, pulling, tearing.

COMPRESSION: crushing, pushing, squeezing.

TRANSVERSE STRAIN: cross strain, bending, deflection.

TORSION: twisting, wrenching.

SHEARING: cutting, nipping; or, when acting along the grain of wood, it is called DETRUSION.

BUCKLING, or crumpling, is a compound strain made up of crushing and bending. Transverse strain may be resolved into tension and compression; and Torsion belongs almost exclusively to machinery. Shearing may generally be taken as equal in amount to tensile strength, and we have then Tension and Compression left, as two chief strains of opposite character, and upon which the others may be said to depend. Now, taking these two strains, we find that the stress per square inch sectional area sufficient to rupture the material when of ordinary quality is as follows:

	Tension.	Compression.
Wrought iron ..	22 tons ..	18 tons
Steel	35 " ..	50 "
Cast iron	7 " ..	42 "

The safe load on a structure is usually taken at $\frac{1}{4}$th the breaking weight, $\frac{1}{4}$th being in that case called the factor of safety, and thus we find that we may safely put the following stresses per square inch sectional area upon the material:

	Tension.	Compression.
Wrought iron ..	5 tons ..	4 tons
Steel	8 " ..	12 "
Cast iron	$1\frac{1}{2}$ " ..	7 "

These are all within $\frac{1}{4}$ of the ultimate strength, the cast iron being from $\frac{1}{5}$ to $\frac{1}{6}$ on account of the irregularities in that material. These must be taken as general or approximate figures, and for dead or gradually applied loads only; as we shall find, in considering special structures, there are specially recognised stresses allowed under the ordinary working loads. Greater loads than these might be applied without endangering a structure, if we could ensure both material and workmanship being thoroughly sound, but unfortunately this cannot be done. If the material be strained to within $\frac{1}{3}$ of its breaking weight it is liable to permanent injury, the atoms being, as it were, pulled so far apart that they cannot get back into their proper places again.

You will observe that wrought iron has nearly the same strength in tension and compression, while cast iron has nearly six times the strength in compression that it has in tension; and hence, in some structures, cast iron is used for the parts in compression, and wrought iron for those in tension.

Now I must go into figures, and those of you who have already some knowledge of the subject must bear with me while I explain to those who are quite beginners, how sectional areas are found. To find the sectional area of a flat or square bar, multiply the breadth by the thickness, and the measurements being taken in inches, the result will be square inches area. For the sectional area of a round bar multiply the diameter by itself and then by 11, and divide by 14. But we can put this into a sort of shorthand, very compact and useful:

$$\text{Square bar } a = b \times t$$
$$\text{Round ,, } a = d \times d \times \tfrac{11}{14}, \text{ or } d^2 \times \cdot 7854.$$

These are formulæ or equations, and the system of putting down letters, instead of figures or descriptions, is neither more nor less than Algebra. If you were to see the section

of an angle iron for the first time, you would doubtless think it a very difficult matter to obtain the sectional area by measurement; so it is, to get it exactly, but it is an easy matter to obtain it approximately. You will see (Fig. 1) that it is curved at the edges and the root, and tapered along the sides; but a straight line parallel with the back, cutting the centre of the tapered side, would give very nearly the same amount of material, and this is how the measurement is taken (Fig. 2), $a = \overline{b \times t} + \overline{(b - t) \times t}$.

Tee iron, channel iron, and cross iron are found in a similar manner. Hollow columns are found by deducting the area of the interior from the area of the exterior, each being found in the way shown for round bars.

We come now to the consideration of a few important principles in Mechanics which must be mastered before we go further. The first of these is the lever. Levers are divided into three orders, according to the relative position of the *power* or active force, the *weight* or passive force or resistance, and the *fulcrum* or pivot round which movement takes place, or tends to take place, thus:

1st order.	Fulcrum	between	the others	(Fig. 3).
2nd order.	Weight	„	„	(Fig. 4).
3rd order.	Power	„	„	(Fig. 5).

But in each of these arrangements there is only one principle involved, and as this is most clearly shown in a lever of the first order, we will take that for investigation. The *lever* is assumed to be a rigid, inflexible body without weight. The *fulcrum* is situated at any point between the two extremities, the *power* is the force or load applied at one end, and the *weight* is the resistance or load at the other end. The lever may be curved or straight, but if curved we take our measurements of its length in a straight line only, measuring from the points of application of the power and fulcrum for the *leverage of the power*, and from the points of

application of the weight and fulcrum for the *leverage of the weight.* The lever may even be assumed to be bent at right angles, the fulcrum being at the angle, but our measurements will be taken in the same way, and there will be no difference in its value as a mechanical power in that respect, provided that the forces are at right angles to each portion of the lever (Fig. 6); but if the forces are not at right angles to the lever, we must take for the true leverage distance a measurement at right angles to the direction of each force, as *ab*, *bc* (Fig. 7). The distance from the power to the fulcrum, as defined in these examples, multiplied by the power, is called the *moment of the power*, and the distance from the weight to the fulcrum multiplied by the weight is called the *moment of the weight.* When these moments are equal the lever balances, but if one be in excess of the other the lever will move until it reaches a position in which it will balance, or until it is stopped by some external object. Suppose the weight to be taken away and the lever prevented from moving by contact with some fixed object, then the pressure against this object will be the same in amount as that of the weight removed. The power may be represented by muscular force or by any external pressure or load, and the weight by any resistance either external or internal. If we call the power P, its leverage x, the weight W, and its leverage y, we can represent the relationship clearly thus:

$$P \times x = W \times y$$

which is called an equation, or the same thing may be explained by "rule of three," thus:

$$P : W :: y : x;$$

and you know that in a proportion sum you multiply the second term by the third and divide by the first in order to get the fourth, thus:

$$\frac{W \times y}{P} = x;$$

but if we have any three of the four given, we can find the remaining one in the same way, thus:

$$\frac{P \times x}{W} = y. \quad \frac{P \times x}{y} = W. \quad \frac{W \times y}{x} = P;$$

and, putting the respective loads in *lbs.*, with their distances in *inches*, in place of these letters, we can obtain the arithmetical result in any given case. Suppose, for instance, we have a case in which the power is 8 lbs., its leverage 12 inches, and the leverage on the other side 10 inches (Fig. 8). Taking $W = \frac{P \times x}{y}$, we have $W = \frac{8 \times 12}{10}$, therefore $W = 9\frac{6}{10}$, or in decimals, $9 \cdot 6$ lbs. as the pressure caused against any resistance, or the amount of actual weight it would support. Exactly the same rules and lettering will apply to the other orders of leverage, and we need not, therefore, take them in detail. There is only one point about which I must caution you particularly, and that is, to take in each case the *full distance to the fulcrum*, which is a point sometimes overlooked in levers of the 2nd and 3rd kinds.

In these calculations the lever is assumed to be a rigid, inflexible bar without weight. The weight of the lever will of course affect the result in any practical case, but by omitting it for the present, the principles will be much easier to understand. To sum up these principles, the product of a force into the perpendicular distance of its direction from any given point is termed the moment of the force about that point. And, taking the power, weight, and resistance at fulcrum, as three pressures by which the lever is kept at rest, the two terminal pressures must always act in the same direction, while the intermediate pressure acts in the opposite direction; the intermediate pressure must always equal the sum of the other two; and, lastly, when equilibrium obtains, the moment of the power about the fulcrum must be equal to the moment of the weight.

QUESTIONS FOR HOME-WORK.

Answers to be given up at next Lecture.

1. Required the breaking weight of a wrought-iron bar $1\frac{1}{2}$ inches by $\frac{1}{2}$ inch, having a load hanging to it. (Ultimate tensile stress.)—Ans. 16·5 tons.

2. Breaking weight of a steel rod $\frac{3}{4}$ inch diameter, with a hanging load?—Ans. 15·46 tons.

3. Safe load on two wrought-iron plates, containing together 9 square inches, and being under direct thrust. (Safe working intensity of stress in compression)?—Ans. 36 tons.

4. Safe load on three steel bars taken together, each being $1\frac{5}{8}$ inch by $\frac{5}{16}$ inch, and acting as ties. (Safe working intensity of stress in tension)?—Ans. 12·19 tons.

5. Safe load on a hollow cylindrical piece of cast iron, 4 inches inside diameter, 6 inches outside diameter?—Ans. 109·96 tons.

6. What sectional area in wrought iron would be required to resist a compressive load of 15·25 tons safely, assuming the piece to be too short to bend under the strain?—Ans. 3·82 square inches.

7. If you assume cast iron to require a factor of safety of $\frac{1}{10}$, what tensile stress could you put upon a piece $7\frac{1}{2}$ inches by $\frac{7}{8}$ inch?—Ans. 4·6 tons.

8. A bar 3 feet 6 inches long, supported at 1 foot 6 inches from one end, has a weight of 18 lbs. hanging on the long end; what pressure will be required at the short end to balance it?—Ans. 24 lbs.

9. A bar 6 feet long is pivoted at one end; what power would be necessary at 1 foot from the free end, to balance a weight of 23 lbs. at 1 foot from the pivot end?—Ans. 4·6 lbs.

10. A bar 4 feet 3 inches long is hinged at one end, it is pushed perpendicularly at a point 1 foot $3\frac{1}{2}$ inches from the free end by a force of 29 cwts.; what pressure will the bar exert against a resistance at the free end?—Ans. 20·19 cwts.

LECTURE II.

Pressure of loaded beams on the supports—Girders and roofs found in same way—Load in centre and out of centre—Parallelogram of forces—Definition and explanation—Composition and resolution of forces—Use in finding effect of load on inclined struts—Horizontal thrust—Combination of parallelograms—Graphic delineation of strains—Leverage by diagrams—Case of similar triangles—Application to pressure of loaded girder on supports—Polygon of forces—Reciprocal diagram—Strain in a strut varies with the angle—Inclined struts—Value of trigonometry—Calculations may be made without it—Horizontal strut—Examples.

We now come to the application of some of the principles we learnt in the first lecture. We will commence with an ordinary girder, finding by leverage what pressure or load is caused upon the wall supporting it. The arrangement will be as shown in Fig. 9, forming virtually a lever of the second or third kind, although it has apparently two fulcrums and no power; but you must bear in mind that these are accidental terms only, and that whether we call any given load a power or a weight, it makes no difference in the result.

First, let us assume the load to be exactly in the centre; it would appear on inspection that the pressure upon each support would be $\frac{1}{2}$ the load $+$ $\frac{1}{2}$ the weight of the girder, and this can be proved to be correct; but if the weight be not exactly in the centre, the pressure will evidently be greater on the nearer support, and in order to find out how much greater, we must make use of our formula for leverage. Here let A be taken as the *fulcrum*, W as the active load or *power*, and B as the passive resistance or *weight*, x the distance of the load from the left-hand support, and l the total distance between the supports, called the span of the girder. Then

the leverage of the power will be x, the leverage of the resistance l, and the pressure on B $= \frac{W x}{l}$. For the pressure on A, we must assume B as the fulcrum, and A the resistance, then the leverage of W will be $l-x$ (i. e. the length of l, less the length of x), and the formula will stand thus:

$$\text{Pressure on A} = \frac{W \times (l-x)}{l};$$

but to each of these answers, *wherever the load may be*, we must add half the weight of the girder, supposing it to be of uniform section, to get the total load on the supports. The proof that the pressure on the supports is equal when the load is in the centre is obtained thus:—When W is in the centre $l - x = x$, and we may therefore put x in place of $l - x$ in the last formula, when it becomes identical with, and therefore equal to, the preceding formula.

If you understand the work so far as we have gone, you have really learnt a great deal; because all girders, roofs, and bridges tied together into one frame follow exactly the same rules. Thus, if this outline (Fig. 10) be taken as representing one truss of a roof, say twenty feet span, and the weight of itself with the covering it supports, including allowance for wind and snow = 5 tons, then the pressure on each support will be $2\frac{1}{2}$ tons. Or, if we take a rigid frame irregularly shaped or loaded, as in Fig. 11, or a lattice girder, Fig. 12, the proportion of the load on each support (apart from weight of girder) will be determined by the formulæ we have just taken.

Now, leaving the question of leverage, we must take up another of the fundamental principles, called the *parallelogram of forces*. A parallelogram is simply a four-sided figure with its opposite sides parallel, Figs. 13 to 16. The principle of the parallelogram of forces is this:—If two adjacent sides of a parallelogram represent by their position

and length the direction and magnitude of two forces, then a diagonal from the junction of these two lines will represent the direction and magnitude of a single force which would have the same effect as the two others combined.

The reverse of this is equally true:—If a force be represented in magnitude and direction by the diagonal of a parallelogram, the two adjacent sides meeting at one end of that diagonal will represent the magnitude and direction of two forces giving a similar result. But the same diagonal would answer for any number of parallelograms, as in Fig. 17, and we can therefore take the most convenient one; for instance, if we have a force acting in the direction A B, Fig. 18, and we wish to substitute for this two other forces, one of which shall be in direction and magnitude equal to A C, then we join C B, make A D parallel to C B, and B D parallel to C A, then A D will be the second force required. Or we may have the force A B, Fig. 18, and have the *directions* of the forces A C A D, but not their magnitude; in that case, by drawing lines parallel to A C, A D, through point B, we obtain their magnitudes.

Now let us apply this knowledge to the case of two inclined struts, thus (Fig. 19), A C and C B, the load upon them being represented by W. Here we have a single force, the load, of given direction and magnitude, and we have the directions of two other forces, or the lines in which they will act, and we want to find their magnitudes. If we mark off on C W a point *w*, so that C equals the load to any given scale, and through *w* draw lines *w a*, *w b*, parallel to B C, A C, then the lengths C *a*, C *b*, will give the respective magnitudes of the forces in C A, C B.

It is clear that these will be pushing forces in the struts, or in other words, the struts will be in compression, and therefore some resistance must be provided to hold their lower ends. We have shown them resting on a wall, and now we must ascertain what the effect is. Here we come to a repetition of the operation we have just performed. We

have, taking one side first, the one force acting in line with the strut to be resolved into two, one acting downwards and giving, as we may say, the weight on the wall, and the other acting horizontally, or tending to cause the foot to slide along the wall, called the horizontal thrust. Mark off from B, B $d = b\ c$, which is the measure of the inclined thrust in the rafter, then draw $d\ e$ vertical, and $d\ f$ horizontal, continue the surface of the wall through B to e and the side of the wall also through B to f, then B e will represent the horizontal thrust, and B f the downward thrust or weight. The downward thrust is of course provided for by the wall, but the horizontal thrust must be counteracted by tying the feet of the struts together, or by bolting them down, unless the bearing surface of the wall be cut away perpendicular to the strut, as in Fig. 20, in which case the thrust and weight would balance each other. Exactly the same description would apply throughout, if the struts were of *different lengths*, the operations being repeated on each side, and the strains being found to be smaller on the side with the flatter angle. Instead of having a separate parallelogram for the foot of the strut, we may combine it with the first one drawn, thus (Fig. 21), and I will take unequal struts this time. C W being set off equal to the weight, $w\ a$ drawn parallel to B C, and $w\ b$ parallel to A C, then a C will be the compression in A C, $a\ c$ the horizontal thrust at A, C c the downward thrust at A, C b the compression in C B, $d\ b$ the horizontal thrust at B, and C d the downward thrust at B.

By comparing this diagram with the preceding one, you will see that this is worked on the same principle, but is more compact. Instead of C b being repeated at the lower end of the strut, it remains at the upper end, and the thrust is resolved by drawing the horizontal and vertical lines from b and C.

In the case of leverage I took the algebraic method of calculation, and in the parallelogram of forces the geometrical method, but they can both be worked out either

way. In any system by which forces or stresses are indicated by the lengths of certain lines, the method is known as "the graphic delineation of strains." If you accustom yourselves to this mode of working, you will be astonished at the number of cases in which it will assist you, and the ease with which complicated questions may be worked out.

It may be useful just to show you how questions of leverage may be worked out with a ruler and pencil. In a lever of the first order we have the arrangement shown in Fig. 22. Over the fulcrum set up w equal in length to the known weight, upon any given scale; join the end of this line with the power end of the lever, from the fulcrum draw a line parallel to the last, and produce a line over the weight to meet it, call this line p. Then, if w represent the weight to be balanced on this lever, p represents the power that will do it; and *vice versa*, if we have a given power and want to find the weight it will balance, we set up p equal to the power and join it to the fulcrum, from power end of lever draw line parallel, then draw vertical line over fulcrum to meet the last, and it will equal W. This method depends upon the properties of similar triangles, or those of different sizes but similar shape. For a full explanation you must refer to Euclid VI. 2, but it is easy to see that it is simply a question of proportion in the triangles:

$$x : y :: W : P$$
$$\text{or, } y : x :: P : W.$$

In levers of the second and third orders, the graphic constructions will be alike, as seen in Figs. 23 and 24.

Set up w over P, equal to the weight (Fig. 23), then line p drawn above the weight will equal the power required; and set up p over W, equal to the power (Fig. 24), then line w, drawn under the power P, will represent the weight that will be balanced.

In representing the pressure of a girder upon its supports

the principle is the same, although the diagram looks rather different. We will take an example, see Fig. 25. Let A B represent the girder whose length is l, and W the load, its position dividing the girder into two portions x and y. Take line A B as a base line, and at each end set up a line w equal to the load W. Then for the load upon bearing A, join point B with the top of line w over A, and through a point on the base line vertically under the W draw a parallel line; the included space a on line w over A, shown thick in the diagram, will then be the proportion of the load borne at A. By reversing the construction and drawing the parallel lines to the right, we get in the same way the portion b representing the load upon B;

$$\text{then} \quad l : x :: w : b$$
$$\text{and} \quad l : y :: w : a.$$

Before we leave the geometrical system for a time I must call your attention to a remarkable property connected with the graphic representation of *forces in equilibrium.* By equilibrium we mean, in plain English, that whatever strain may be on the different parts, there is no tendency to move unless something gives way. The property is this: if the forces be represented by lines of similar direction and proportional magnitude, they will form, if put together as a continuous outline, a closed figure of as many sides as there are forces: thus, if Fig. 26 represent three forces in equilibrium, the lines put together will make Fig. 27, called a *triangle of forces*; or if there are more than three forces, as in Fig. 28, the lines put together will make Fig. 29, called a *polygon of forces.* If the lines be drawn at the proper angle and of the right length, but do not form a closed figure, then we know for a certainty that the forces are not in equilibrium, and that we have over estimated—or under estimated—one or more of them, and hence this mode of re-arranging the lines forms a valuable check upon accuracy of the work. The

converse of the polygon of forces is equally true, viz.: If the sides of any closed figure be taken to represent the directions of the same number of forces, taken in order, and meeting in a point, then the lengths of the sides will represent the magnitudes of the forces. This is an exceedingly useful property, and we shall find that it enables us, by means of what is called a "Reciprocal diagram," to ascertain easily the strains upon a complicated framework.

We have seen how to find the strains on inclined struts geometrically; it is also desirable to know how to find the same by calculation. It is evident that if the strut were vertical, the strain would be exactly equal to the load; while if it were horizontal, there would be no strain at all (the end not being fixed), the difference between these two extremes depending upon the angle of the strut. If, however, we make our calculations from the angle itself, we shall have to use trigonometry, which means three-angle measurement; but we can avoid this by calculating from the parts which form the angle, viz., the height and the span.

Let C D, Fig. 30, represent the proportion of weight borne by the strut C B, then the length C B will represent the compressive strain in C B. The calculation is very simple. Let us mark the perpendicular height C D = P, the inclined length C B or hypothenuse of triangle B D C = H, and the horizontal distance B D or base of triangle = B, w = proportion of load borne by strut C B as found by leverage, t = horizontal thrust at foot, d = downward thrust, and s = inclined thrust or strain in the strut or rafter; then,

$$\mathsf{P} : w :: \mathsf{H} : s, \text{ or } s = \frac{w\,\mathsf{H}}{\mathsf{P}}$$

$$\mathsf{P} : w :: \mathsf{B} : t, \text{ or } t = \frac{w\,\mathsf{B}}{\mathsf{P}}$$

$$\mathsf{H} : s :: \mathsf{P} : d, \text{ or } d = \frac{s\,\mathsf{P}}{\mathsf{H}}.$$

These simple formulæ being due to the fact that the length of the sides represents proportionally the strain in each.

If out of three dimensions—base, perpendicular, and hypothenuse—you have only two given in figures, you can measure the third off to scale, which is quite near enough for ordinary purposes; but if you should wish to have mathematical accuracy, the following will be the formulæ:—

$$B = \sqrt{H^2 - P^2}, \quad P = \sqrt{H^2 - B^2}, \quad H = \sqrt{B^2 + P^2}$$

depending upon Euclid I. 47.

Now suppose one of the struts to be inclined and one horizontal, as in Fig. 31, the horizontal strut will have no direct strain from the load, but it will have a great strain due to supporting the end of the inclined strut. A parallelogram with C w = full load, according to any given scale, will give us at once C a = strain in C A, and C b strain in C B.

By calculation the strain in A C = $\frac{B\,W}{P}$, and that in C B = $\frac{H\,W.}{P}$

In all these illustrations we have omitted the weight of the parts, because it would unnecessarily complicate the questions until we have taken up distributed loads, which we shall do in the next lecture.

QUESTIONS FOR HOME-WORK.

1. A girder 24 feet long, with a span of 20 feet, has a load of 9 tons 3 feet away from the centre; what will be the pressure on the nearest support, omitting the weight of the girder?—Ans. 5·85 tons.

2. A lattice girder weighing 1 ton carries a uniformly distributed load of 13 tons over a span of 11 feet; what proportion of the total load will be borne by each of the supports?—Ans. One half on each = 7 tons.

3. An inclined rafter 8 feet long rests against another 15 feet long, and the lower ends are supported on a level surface; show the amount

and direction of the various strains set up when a load of 9 tons is carried at the apex, and the feet are 18 feet apart?—Ans. See Fig. 32.

4. A cast-iron girder 17 feet 3 inches span has a load of $7\frac{3}{4}$ tons resting at a point 5 feet 9 inches from support A; what will be the load on support B?—Ans. 2·58 tons.

5. The upper end of a rafter inclined at an angle of 45° rests against a horizontal strut; what weight must be hung at the junction to balance an upward force of 2 tons acting along the inclined rafter?—Ans. $\sqrt{2} = 1 \cdot 414$ tons.

6. In the last question, if the rafter were placed at a steeper angle with the same load, would the strain in it be greater or less?—Ans. Less.

7. Two forces of 10 and 15 tons respectively act upon a point at right angles to each other; if the direction of the smaller force should be altered to 120° from the greater, what must be their magnitude to produce the same resultant as before?—Ans. 11·55 and 20·77 tons.

LECTURE III.

Distributed load carried on horizontal beam—When inclined, strain varies with angle of bearing surface on support—Lean-to roof—Strain in pair of inclined rafters—Span roof—Tie rod—King bolt—Trussed beam—Strains in same—Strength of girder according to method of fixing and position of load—Cantilevers—Ordinary girders—Continuous girders—Typical form of cross-section of girder—Action of load on a cantilever—Distribution of the strains—Investigation of leverage—Construction of formulæ—Diagram of strains; tension, compression, and shearing—Strain at intermediate points found by ordinates—Examples.

We have, so far, considered the effect of concentrated loads only; if we had assumed the loads to be spread over the surface, or distributed, it would have modified some of the results. These cases require careful attention, because they nearly always involve more difficulty than concentrated loads. Let us take an easy one first—an ordinary girder or horizontal beam, as in Fig. 33. As it stands, each support would bear half the full load, but if the beam were inclined what would be the result? It will vary according to the nature of the bearing at the higher end, A, Fig. 34; suppose we can secure a horizontal bearing, as in Fig. 35, then the load will be supported half at A and half at B, each half resting on the support, and acting as downward thrust or weight, without putting any compression on the rafter, or thrust at the foot. There will of course be a cross strain, tending to break the rafter by bending, but that is a matter for separate consideration. If the higher support were placed vertical instead of horizontal, as in Fig. 36, then point A would get no direct support, and there would be the same pressure against the wall as if it were

a horizontal strut. Taking for weight the half of the load that ought to be supported at A, the effect of this half is transmitted down the rafter to B, where it is resolved into a downward thrust and a horizontal thrust, as in Figs. 30 and 31. It will be found that this downward thrust is just equal to half the total load, while the other half is resting direct upon B; therefore B gets the whole load upon it, besides the portion found as horizontal thrust, which will just equal the thrust found upon the vertical wall. This is the condition of things in the simplest form of roof, viz. a lean-to, and an intelligent carpenter will always give a horizontal bearing like Fig. 35, to prevent any outward thrust, although it is not at all unusual to find it omitted. If instead of the vertical wall we add another rafter, inclined the opposite way, we shall have the conditions of a common span roof or pent roof, as in Fig. 37. Here we have each rafter supporting $\frac{1}{2}$ the load, say $\frac{1}{4}$ resting at the lower ends, and $\frac{1}{4}$ at the upper ends, the $\frac{2}{4}$ at the upper ends reacting against each other, causing compressive strains in the rafters, which will be resolved at A and B into thrust and weight as before. The $\frac{1}{4}$ at each lower end is simply dead weight on the supports; and it is most important to observe that, omitting the cross strain, no effect whatever is produced by it upon the rafters. The thrust from a roof of this kind frequently causes the supports to be pushed over, and hence it is only used for small sheds and greenhouses. The thrust may be met by tying the ends of the rafters together, as in Fig. 38. This tie rod will be in tension because the ends tend to be forced asunder. The amount of strain will equal the thrust found *on either side*, not the two thrusts added together, because they are acting in opposite directions. An illustration would be, two men pulling at a rope in opposite directions; although two men would be pulling, there would only be what might be called a one-man strain in the rope, because the other man might be replaced by a dead post.

In this sketch we have the simplest form of roof truss, but there is a practical defect in it, apart from decreasing the head-room for walking underneath. You know that it is impossible to pull a chain or string to a perfectly straight line: it bends down in the middle, however hard it may be pulled; this bending is called sagging, and to prevent the sagging of the tie rod in the example before you a king bolt is often put in, as in Fig. 39, to hold it up in the middle. In calculating the strains on this as a roof truss, you will find no strain on the king bolt due to the load on the roof; all it has to do is to bear a portion of the weight of the tie rod; we are, however, for the present, omitting all strains due to the weight of the parts, and also the effect of cross strains.

If we reverse the last example, as in Fig. 40, we have the simplest form of a trussed girder or bridge; it will be familiar to you as the method of strengthening a beam of timber for carrying loads. In this arrangement all the strains are reversed: we have C D now acting as a strut in compression, and A D, D B, as ties in tension, A B being in compression. Suppose we had a load distributed over the beam, there would be one half on A C, and the other half on C B. The half on A C would be resting half of it on A, and half on C; while that on C B would be resting half on C, and half on B; so that we should have $\frac{1}{4}$ the full load on A, $\frac{2}{4}$ on C, and $\frac{1}{4}$ on B. The portions at A and B would rest direct upon the support, without causing any strain to pass through the framing, but the $\frac{2}{4}$ on C would be transmitted through C D, and thence through D A, D B, to A and B; because, whatever the arrangement of trussing, the final result comes to the abutments or supports. C D being exactly in the direction of the load on C, that load, viz. $\frac{1}{2}$ the total load, will be the measure of the strain on C D without requiring any calculation. By drawing the parallelogram shown in Fig. 41, the load *c* D will be resolved into

two forces, D *a*, D *b*, giving the tensional strains in D A, D B. Then by joining *a b* we resolve the force *a* D into *a o*, *o* D, and *b* D into *b o*, *o* D, the horizontal lines *a o* or *o b* giving the compression in A C B; and the vertical line *o* D, the weight on A and B, in addition to the portion of the load which we found resting there direct.

We might continue to take other forms of trussing, more and more complex, and work them out in the same way, but before we proceed further in this direction it will be necessary for us to learn something of the effect of cross strains, such as occur in girders.

The effect of the load upon a girder, or any piece of a structure subject to cross strain, varies considerably, according to the manner in which the girder is supported and the nature of the load. I might give you a tabulated statement of this, but it will be better to work it out and show you the reason of these differences.

First take a girder having one end built into a wall, or otherwise firmly fixed, as in Fig. 42, and called in this position a cantilever; or the projecting end of a girder beyond a column would be a similar case. Suppose this girder to be 12 feet long, and just strong enough to bear a load of 1 ton on the end; then if the load be distributed, as in Fig. 43, it will bear 2 tons, because one half will be carried direct by the support, and the other half or 1 ton will be carried by the end of the cantilever as before. Or we may look at it another way, and say that if the load were collected at the centre of gravity it would have a leverage of only half the amount, the moment of 2 tons at 6 feet being equal to the moment of 1 ton at 12 feet, and therefore of corresponding effect as regards the maximum strain occasioned in the girder, although we shall find further on that the detailed effect differs.

Now, if we can support the ends of the girder, leaving the same clear span as before, and put a concentrated load in

the middle (Fig. 44) it will bear 4 tons, because we may say 2 tons is on the middle end of each half of the girder, and as the half girder is only half the span or leverage of the previous example, the cantilever, it will bear double the load with the same strain. If the load be spread over the girder as in Fig. 45, it will carry 8 tons; we may say 4 tons is on each half of the girder—2 tons on the supports direct, and 2 tons on each side at the centre—equivalent to the previous 4 tons in the centre. If both ends of the girder be built into a wall for some distance, or otherwise firmly secured at the ends, as in Fig. 46, we may put 8 tons in the centre. We may look upon this as two cantilevers, each of one-fourth the span of our former one, and supporting at their ends a girder of half the span, as in Fig. 47. The cantilevers being one-fourth the span will each carry 4 times the load ($1 \times 4 = 4$, $4 \times 2 = 8$ tons). But suppose the load to be distributed, this girder will presumably carry 16 tons, because half will be carried by the walls direct, the other half straining the girder as before. Professor Unwin says that a girder under these conditions would only carry 12 tons; Dr. Anderson says it is a doubtful case, and that different authorities give from 12 to 16 tons: so that we may look upon 12 tons as the minimum to allow, and 16 as the maximum. Taking the latter figure, and reviewing our illustrations, we find that similar girders will carry sixteen times as much under one set of conditions, as they will under another set. This is a particularly instructive set of examples, because it will show you that the dimensions and weight of ironwork are not altogether dependent upon stresses, but that there is in any structure plenty of scope for good designing. You will find the girder with fixed ends commonly occurring as part of a continuous girder: that is, one continued over several supports, as in the railway bridges over the Thames.

The typical form of a girder, taking a cross section, is shown in Fig. 48, consisting, as you see, of a top and bottom flange kept apart by a web, and prevented from twisting by

means of angle irons in the case of wrought-iron girders, and by casting in one piece in the case of cast-iron girders. The flanges and the web are the main parts; angle irons, stiffeners, cover plates, rivets, &c., being mere details of construction.

We have considered the loading of different girders from what we may call a common-sense point of view, which we shall find very useful presently, but we must look a little closer into the effect of these loads; the simplest case will be that of our first example, the cantilever, loaded at the end, Fig. 49.

It is clear that the load will tend to cause the girder to bend down, as in Fig. 50, and it is also clear that this bending strain is made up of a stretching or tension in the top flange, and a crushing or compression in the bottom flange. The web is not considered to render any assistance in resisting these strains, its sole duty being to keep the flanges apart, but there is a strain in it due to the direct action of the load which will claim our consideration. We may look upon the bending strain as simply a question of leverage, and there are three ways of considering it, all coming to one result, Figs. 51, 52, and 53.

The load multiplied by the length of the cantilever = the moment of the power in each case. Then for the moment of the weight we have in the first case the resistance in the top flange $\times \frac{1}{2}$ the depth, + the resistance in the bottom flange $\times \frac{1}{2}$ the depth. In the second case, the moment of the weight will be the resistance in top flange $\times$ whole depth, and in the third case resistance in bottom flange $\times$ whole depth. Putting letters for words, we have these equations:

$$\text{I. } W \times l = (t \times \tfrac{1}{2} d) + (c \times \tfrac{1}{2} d)$$
$$\text{II. } W \times l = t \times d$$
$$\text{III. } W \times l = c \times d;$$

or putting the same statements as formulæ:

$$\text{I.}\quad t + c = \frac{W \times l}{\frac{1}{2} d}$$

$$\text{II.}\quad t = \frac{W \times l}{d}$$

$$\text{III.}\quad c = \frac{W \times l}{d};$$

so that, whichever way we consider it, the strain is equal in each flange. To take an example, say—load 4 tons, length 8 feet, depth 2 feet, strain in either flange $= \frac{4 \times 8}{2} = 16$ tons. This is of course the strain next to the wall, where the full length of the cantilever comes into action. If we want the strain at any immediate point x, Fig. 54, we take the horizontal distance W x as leverage of the power, and by working out a few points we shall find that the strain along the girder is exactly proportional to the distance from the lead, and that immediately under the load, as there is no leverage, so there would be no strain.

This can be shown geometrically in a beautifully simple manner. If we set up A a to any given scale, making it equal to the maximum strain as found by the calculation, and then join a with the point under W, the vertical height or ordinate from the base, at any point x, will give the amount of strain in the flanges of the girder at that point, being tension in the top flange, and compression in the bottom flange. The whole thing may be done geometrically if required, thus, Fig. 55: put the girder down to scale, join the top of the girder b at one extremity with the bottom c at the other extremity, produce the line of underside of girder A c indefinitely, set up A d = W, draw $d\,e$ parallel to $b\,c$, from point A with radius A e describe an arc, and continue A d to

meet it in f, then $A f$ will equal the result found by the previous calculation, viz. the maximum strain on the girder. The advantage of the geometrical method is that it can be used just as readily if the question be full of fractions, but the result is of course only approximate, depending upon the neatness in working. If you are careful to use a good scale, and a hard pencil with a fine point, the result will be near enough for all practical purposes.

I mentioned that there was also a strain due to the direct action of the load; this is a shearing strain, and is assumed to be borne entirely by the web. It is a difficult mathematical investigation to prove what part the flanges and web respectively take in resisting the various strains, but it is found to be practically correct to assume that the tension and compression are taken by the flanges, and the shearing by the web. The tendency of a shearing strain is to cause one section of the girder to slide on another, as if cut or sheared, as shown in Fig. 56.

In the case of a cantilever loaded at the end, the shearing strain is uniform throughout, except for the slight alteration made by the weight of the girder itself, and is equal to the load. If the load be 20 tons, the shearing strain will be 20 tons, or allowing 4 tons per square inch, the sectional area of the web must be 5 square inches, and uniform throughout its length. The shearing strain would be indicated geometrically, as in Fig. 57. Drop line $A b = W$, draw $W c$ vertically, and $b c$ horizontally, then ordinates from base line of girder to $b c$ give the strain at any point. All the ordinates being equal, the strain is seen to be uniform.

QUESTIONS FOR HOME-WORK.

1. A rafter 11 feet long, with one end 5 feet 8 inches higher than the other, has a distributed load of $2\frac{3}{4}$ tons; what will be the horizontal thrust at each end, and what the load upon the wall at the foot?—Ans. Thrust 2·28 tons, load 2·75 tons.

2. A trussed beam 24 feet span has a concentrated load upon it of 10 tons, 8 feet from one end, the strut is directly under the load, and 12 feet long; what will be the amount and nature of the strain in each portion?—Ans. See Fig. 58.

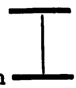
3. A tall man and a short man have a load slung on a pole midway between them; which will bear the greater proportion of the load, and why?—Ans. The bearings will be in line with pole (see Fig. 59), the load on each man will be $\frac{W\,B}{2\,H} = \frac{1}{2}\,a$. There will also be a sliding tendency to be stopped by friction, amounting to $\frac{W\,P}{2\,H} = \frac{1}{2}\,b$ on each man. But there will be a tendency for the load to work downwards towards the short man, who will then have to bear the greater proportion.

4. If a cantilever carry a load of $7\frac{3}{4}$ tons safely at its extremity, how much would it carry evenly distributed?—Ans. $15\frac{1}{2}$ tons.

5. If a girder supported at both ends just break with a distributed load of 27 tons, what load placed in the middle would cause its fracture?—Ans. $13\frac{1}{2}$ tons.

6. Sketch a typical form of girder in section, and name the strains in each part when fixed as a cantilever and loaded in the middle.

Ans. Top flange, tension .. ..
Web, shearing .. .. ..
Bottom flange, compression



LECTURE IV.

Effect of distributed load—Strain varies as the ordinates to a parabola—Setting out parabola—Method when particular points only are required—General calculation of strains in a girder divided into three stages: pressure on supports, strain in flanges, strain in web—Girder with concentrated load in any position—Maximum strain immediately under load—Variation of strain between extremities—Investigation by diagrams and formulæ—Also by leverage—Combination of diagrams from two or more loads—Special diagrams for rapid use—Girder with distributed load, method of finding strains—Proportioning material to resist the strains—Examples.

If a cantilever be loaded with two or more weights, we can find the collective result by taking each result separately, in the manner shown in the last lecture, and then adding them together. The extreme case will be when the cantilever is loaded uniformly throughout its length. We shall find the *maximum* strain only one-half of what it is with an equal load concentrated at the outer end; the effect is the same as if the whole of the load were collected at its centre of gravity, which would be half way along the cantilever, and have therefore only half the leverage, as in Fig. 60. The intermediate effects will, however, not be similar. If we calculate the result of each of these small weights at its respective leverage, we shall find that we do not obtain a uniform difference of strain along the girder as before. Putting down the heights found for the strain at the various points, we shall find that they form a curve instead of a straight line. This curve will be one-half of what is called a parabola, with the vertex at end of cantilever, and, as it is quite indispensable to be able to construct one of the right shape and any given size, I will tell you how to do

it. Let A, B, Fig. 61, be the length, and A C the height; divide A B into any number of equal parts, and A C into the same number. Set up B D equal and parallel to A C, join C D, and from each point on A B draw a vertical line parallel with A C. Then join B with each of the points on A C, and through the intersection of the lines from similar points on A C, A B, draw the curve C B. Then, ordinates or vertical measurements from any point on A B to the parabolic curve will give the strain at that point.

But suppose we only want to know the strain at one or two particular points, we need not draw the whole curve, thus: required the strain at A, x, and y, Fig. 62. We will divide this into two diagrams to show the separate stages more clearly, although in practice you would work right through on the one figure. First, for the strain at A, this will be equal to the height of the parabola and may be found geometrically. Take A B as the base line, Fig. 63, and g as the point under the centre of gravity of the load, then A g will be the leverage of the load. Let A c = depth of girder, join $c\,g$, set up A d = load to any given scale, and from d draw $d\,e$ parallel to $c\,g$, then from point A with radius A e describe an arc cutting A d produced in point f. Then A f will be the strain in the flanges at point A. Now for the strains at x and y, see Fig. 64: set up A f from the last figure, and B h equal and parallel to it, join $f\,h$, and draw vertical lines from x and y to meet $f\,h$ in points m and n. Join A h and draw $m\,o$, $n\,p$, parallel to A h; join o B, p B, then where o B cuts $m\,x$ call the point q, and where p B cuts $n\,y$ call the point r. Then $x\,q$ will be the measure of the strain in the flanges at x, and $y\,r$ the strain at y.

With a cantilever loaded in this manner, the shearing strain, instead of being uniformly equal, is uniformly reduced, being equal to the total weight at the support end, and tapering off to nothing at the outer end, as in Fig. 65, the vertical height at any point being the measure of the shearing strain at that point. You will observe that shearing strains are usually

shown below the girder, while flange strains are shown above it, using the bottom of the girder as a base line.

Our investigation of the strains in a cantilever will help us to understand more readily those in an ordinary girder supported at each end. First, with a concentrated load, Fig. 66; we may divide the calculations into three stages, viz.: the pressure on the supports, the strain in the flanges, and the strain in the web. We have seen already how to find the pressure on the supports, but I will show you a slight modification of the same principle, which will also be useful as leading up to the discovery of the strains in the flanges.

Let A B, Fig. 67, be the girder with span l, and W the position of the load, dividing the span into the portions x and y; take a large scale, as there are several lines to put on. Set up B c = the load on the given scale, join A c, then the vertical height between the bottom of the girder under the the load, and this line, viz. $e\,d$, will be the proportion of the load borne by support at B. Draw $c\,f$ parallel to girder, and produce $e\,d$ to f, then $d\,f$ will be the load on support A, the whole load $e\,f$ being divided between the supports A and B in the proportions $e\,d$, $d\,f$.

Now $e\,d$ being the load on B, we may look upon the resistance, which is of course equal to the pressure, as an active reaction or power, with the leverage y, and we get the following proportion:—As depth of girder : leverage : : reaction at support : strain in flange. I will give you a little sketch, as soon as we have done with the present one, which will put this proportion in a very clear light.

To resume our construction, let $e\,g$ be the depth of the girder, then join g B and from d draw $d\,h$ parallel to g B; then eh will equal the strain because

$$eg \;:\; e\text{B} \;::\; ed \;:\; eh.$$

From e as a centre, with radius $e\,h$ describe the quadrant $h\,i$, join i A, i B; then the strain in the flange, at any point, will be obtained by ordinates from A B to A i B.

If we had taken support A as our starting point, we should have worked in a similar manner, and have arrived at precisely the same result; so that it does not matter at all which side we take for the flange strains, but it does matter as regards the shearing strain, as I shall now proceed to show you. In the case of a concentrated load the shearing strain is uniform throughout, when the load is central; but when the load is out of the centre, the shearing strain is measured on each side by the proportion of the weight which is carried by the support. To indicate this graphically we must make some additions to our sketch. Drop $A j = W$, join j B intersecting $i\ e$ produced in k, draw $k\ m$ vertical and $j\ m$ horizontal, then $e\ k$ will be the shearing strain on side A, and $k\ m$ the shearing strain on side B; but we must show it on the sketch a little more clearly. Draw $m\ n$ parallel to j B, draw horizontal lines $n\ o$ and $k\ p$, then the shearing strain at any point will be given by ordinates to the outline A $p\ k\ o\ n$ B.

Now for the sketch that is to explain in a very simple way the strength of a girder by leverage (Fig. 68.) Here we have two levers joined at right angles to each other at the fulcrum, and shown by thick lines; viz., the length from load to support, and the depth. Then we have the load which gives us the *position* of the fulcrum, the reaction at B which is influenced by the amount and position of the load, and S the resistance in either flange under the load, say the compressive stress in the top flange. From these particulars we obtain the following proportions:

$$l : x :: W : B$$
$$d : y :: B : S.$$

Putting the latter into an equation, we have $S = \frac{B \times y}{d}$ but B is made up of $\frac{W \times x}{l}$ as shown by the former proportion, and as we found in considering leverages; we may

therefore rub out the B, and replace it by $\frac{W \times x}{l}$ thus:

$$S = \frac{\frac{W \times x}{l} \times y}{d},$$

and the same thing in a simpler form will be

$$S = \frac{W \times x \times y}{l \times d},$$

but in a formula of this kind it is unnecessary to put the sign of multiplication; if the letters are simply written against each other, thus,

$$S = \frac{W\,x\,y}{l\,d},$$

it is understood that those above the line are multiplied together, and those below the line are multiplied together, the line indicating that the upper group is divided by the lower group.

Algebraically, the shearing strain on either side of the load is found in the same way as the pressure on the supports, viz.:

$$\text{Side B} = \frac{W\,x}{l}, \text{ side A} = \frac{W\,y}{l}.$$

If there are two or more concentrated loads, the strains due to each must be worked out separately and added together. If they are worked out geometrically, a separate diagram is made, upon the same foundation, for each load, and the two diagrams are combined to form a final diagram. In adopting this method care must be taken that any portion of the second series of strains overlapping the first is added on to the outside, as in Fig. 69. Take A B as a girder supported at the ends, *c d* as the position of two loads, and A *e* B, A *f* B, the strains in the flanges produced by the respective loads. The two diagrams overlap at the shaded

part A $g\,h\,i$ B. Add $c\,g$ above $e = e\,j$, di above $f = f\,k$, then join A $j\,k$ B which will represent the combined diagram from the two loads.

In finding the strain from a *distributed load* by leverage, you must bear in mind that the reaction to be taken at the support is only half the load on the support, as previously explained, or one-fourth of the total load, the other one-fourth upon the support resting there direct, without affecting the maximum strain on the girder (see Fig. 70). If this point be remembered, the leverage mode is by far the better way to work out an examination question, because you can reason it all through without being dependent upon your recollection of formulæ.

The form of diagram which I have selected in each case has been that in which you can trace the reason for the different lines, in order to assist you to think out the combinations in other examples. There are some simpler forms very readily applied, and depending upon the principles you have already learnt; but not so self-evident. Here is one (Fig. 71), giving all the strains in a girder carrying a single concentrated load. From A drop a vertical line A c = the weight, on any given scale; join B c, drop a vertical line from W cutting B c in d, from d draw $d\,e\,f$ horizontally, cutting A c in e, join W A and produce the line to meet $d\,e\,f$ in point f. As a check upon the working, $f\,c$ ought to be found parallel with W B. Then A e will be the load on A, $e\,c$ the load on B, A e will also be the shearing strain between A and W, $e\,c$ that between B and W, and $e\,f$ will be the maximum strain in the flanges under the load, giving tension in bottom flange and compression in top.

If there were a second load, the strains from it might be worked out at the other end of the girder, and the two sets combined on the top.

There is an alternative method of working this diagram which I will give you with the same lettering (Fig. 72). I

have departed from the usual method of taking a central load for consideration first, and then assuming it out of the centre, because it is apt to mislead and cause you to think that different methods must be adopted according to the position of the load, whereas one method answers all purposes.

In all these diagrams you will observe that, although the girders are supposed to be drawn to scale, I terminate them at the edges of the supports, because, theoretically, that includes the whole of the portion under strain.

There is sometimes shown an extension of the last diagram, by which the strain is found at any point, as well as the maximum strain; see Fig. 73. On a girder A B C D, with load at W, set up A e = load on A, and Bf = load on B, produce A W to meet a horizontal line from e in point g, and B W to meet line from f in point h. Then, to find the strain at any point in the top flange between C and W, draw line from A through the point to meet line $e g$, and the distance from e along the line $e g$ will be the strain. Also for the strain between W and D, draw line from B through the point required to meet $f h$, and the distance from f will be the measure of the strain. The maximum strain will be either $e g$ or $f h$, which will be equal to each other. The strains in the bottom flange will of course be equal to those in the top flange, at the same horizontal distances from the abutments. This is a neat diagram, but on the whole is not so useful for practice as one giving the maximum strain, which is then placed over the load, and lines drawn to the supports, as in the large one (Fig. 67), given this evening. The reason will be seen when we look at the mode practised by draughtsmen for finding the proper lengths of the plates in a girder.

Suppose the girder to have a concentrated load, and the maximum strain requires that four $\frac{5}{8}$-inch plates shall make up the thickness just under the load; we set off the four

$\frac{5}{8}$-inch plates full size at the point of maximum strain in the flanges A B, and join the highest point with the extremities. Then we draw the plates so far that they overlap the "strain-line" just far enough to allow of the rivets securing the ends, as in Fig. 74.

The due proportion of the amount of material in the different parts of a girder belongs to "practical designing," such as you will find exemplified in a series of handbooks to be published shortly.* At present we must confine ourselves to finding the strains, and there are a few other cases of transverse strain which we must consider before we return to framed structures. We shall complete this portion of our subject in the next lecture.

QUESTIONS FOR HOME-WORK.

1. A cantilever of wrought iron is loaded with a weight of 3 tons at the end, and 4 tons spread uniformly along it; the length is 7 feet and the depth 1 foot; show how all the strains are obtained, and state their amount at a point 3 feet from the support.—Ans. (Diagram not drawn):

From end load, tension and compression	12·00
" distrib. " " " "	4·57
Total	16·57
From end load, shearing	3·00
" distrib. " "	2·28
Total	5·28

2. Work out geometrically all the strains in a girder 20 feet span, 1 foot 8 inches deep, with a concentrated load of 5 tons 5 feet from one

* 'Designing Wrought and Cast-iron Structures,' by Henry Adams. Part I., Wrought and Cast-iron Girders, now ready.

end, and prove the result algebraically, excluding the weight of the girder itself?—Ans. (Diagram not drawn):

Flanges, under load	=	11·25 tons.
Load on A	=	3·75 „
Shearing, W A	=	3·75 „
Load on B	=	1·25 „
Shearing, W B..	=	1·25 „

3. A girder, 20 feet span, 20 inches deep, has a load of 3 tons at 5 feet from one end, and 2 tons at 5 feet from the other end; what is the nature and amount of the strain in the top flange at the centre of the span, excluding weight of girder?—Ans. 7·5 tons, compression.

4. In the last case, what additional strain would be caused if the weight of the girder (30 cwt.) were taken into account?—Ans. 2·25 tons.

LECTURE V.

Girder with distributed load, other forms of diagram—Construction of ordinary formulæ—Girders with intermediate supports—Continuous girders—Proportion of spans—Action of load—Points of contrary flexure—Diagram of strains—Formulæ—Humber's Handy Book, recommended for general reference—Moving loads—Moving loads on cast-iron girders—Factor of safety—Formula for cast-iron bridge girders—Strength of beams, ⌊ and ⊤ irons—Transverse strains on rafters—Struts, stanchions, and columns—Joints and fastenings—Rivets, pins, and bolts—Single or double shear—Examples.

THIS evening we have to finish the cases of transverse strain. We will commence with a girder under a uniformly distributed load, an exceedingly common case, and I will show you how the formula in "Molesworth" and elsewhere is made up. For the load on each support (see Fig. 75) we take one-half the total load: this you have seen before. For the strain in the flanges, it is evident that the greatest strain would be in the centre at e. In finding the amount of the strain, we may say that half the load on e D is resting direct upon B, and the other half, or one-fourth the total load, is producing the strain, with the leverage e D against the depth B D, as in Fig. 70, then

$$\text{B D} : e\text{D} :: \tfrac{1}{4}\,\text{W} : \text{strain},$$

but e D $= \frac{1}{2}$ length, say $\frac{1}{2}\,l$; and D B = depth, say d; therefore,

$$d : \tfrac{1}{2}\,l :: \tfrac{1}{4}\,\text{W} : \text{strain},$$

or

$$\text{S} = \frac{\frac{1}{4}\text{W} \times \frac{1}{2}\,l}{d},$$

or, multiplying by $\frac{1}{4}$ and dividing by 4, being the same thing, we may get rid of fractions, thus:

$$S = \frac{W \times \frac{1}{2} l}{4 d},$$

and again, dividing by 2 instead of multiplying by $\frac{1}{2}$, we get

$$S = \frac{W l}{8 d},$$

which is the ordinary formula for the strain in the centre under a distributed load.

To find the strain at any other point *m*, Fig. 76, the formula will be $S_m = \frac{W x y}{2 l d}$, and if the load be given at per foot run $= w$, we take the whole load as $w l$, so that our formulæ become

$$S = \frac{w l^2}{8 d}, \quad \text{and } S_m = \frac{w x y}{2 d}.$$

To obtain the strains geometrically, draw your girder to scale, say A B, Fig. 77, with depth *c d*, set up a centre line at *c*, and mark off *c w* = *one-fourth* the total load, join *d* B, and parallel to it draw *w e*. From *c* as a centre describe the quadrant *e f*, then *c f* will be the flange strain in the centre. Next set up A *g*, and draw *f g* horizontally to meet it in *g*, mark off A *c* into any number of equal parts, and A *g* into the same number, draw vertical lines from the points on A *c*, and cut them by lines drawn from *f* to the divisions on A *g*; these points of intersection will give half a parabola with its vertex at *f*, and the height to the curve at any point will equal the strain. We need not draw the other half of the parabola, as the strains will be exactly similar on both sides of the centre line. For the shearing strain join B *w*, and produce it to cut A *g* in *h*, join *h c*, then vertical

measurements from A c to h c will give the shearing strain at any point. You will find that it is equal at the supports to half the load, and disappears at the centre.

The rapid diagram shown in the last lecture for a concentrated load will apply, with a slight modification, to the case of a distributed load.

Let A B, Fig. 78, be the girder, with depth $c\,d$. Drop A e = $\frac{1}{2}$ W (W = total load), join e B, drop $c\,d$ to f, join c A and produce the line, draw $f\,g$ horizontally, and produce to cut line through c A in point h; join $d\,e$. Then $g\,h$ will be the maximum strain in centre of flanges, A e the load on each support, and ordinates from A d to $e\,d$ will give the shearing strain in the web.

There is an arrangement adopted in some bridges by which lighter girders can be used, and that is by putting in intermediate supports, or we may say the girder is continuous over so-many spans, and therefore generally called a continuous girder. Suppose we have one continued over three spans, as in Fig. 79, the outer ones being preferably about $\frac{3}{4}$ths the length of the middle span. The tendency of the girder under a distributed load will be to bend, as in Fig. 80, the thick lines showing the parts in compression, the thin lines those in tension, and the effect will be the same as if there were three girders fixed firmly at their ends over the intermediate piers.

The points in the flanges where the strain changes from compression to tension, and *vice versâ*, are called the points of contrary flexure, or in plain English, the points where the bending reverses. A graphic diagram of the strains obtained from formulæ will show, easily and clearly, both their nature and amount. Taking w as the load per unit of length, and the girder as of uniform strength, set up E F in the centre of span a, Fig. 81, = $\frac{w\,a^2}{8\,d}$, and construct a parabola

A F B. Then at B erect B G equal to $\frac{w\,a^2}{6\,d}$. Join A G intersecting A F B in H. Then H will be vertically over the point of contrary flexure, A F H will give the compressive strain in top flange and H G B tensile strain in top flange, measuring vertically on the shaded part of the diagram.

At centre of span B C set up I J = $\frac{w\,b^2}{8\,d}$, and construct parabola B J C, set up B G and C K each = $\frac{3\,w\,b^2}{3\,2\,d}$; but if this value be less than that formerly found for B G, then use the former value, if greater use the present and alter line A G accordingly. Then join G K, intersecting B J C in L M. B G L will give tensile strain in top flange, L J M compressive strain in top flange, and the remainder of the girder will be similarly strained to the parts already taken. The strains in bottom flange will be equal throughout to those in top flange, but of opposite character.

There are of course very many varied conditions of load, construction, and support, in the girders that you meet with in practice; it would take too long to investigate the whole of them here, but I think I have told you enough about this portion of the subject to enable you to work out for yourselves other cases, or at any rate to understand the working out from such a book as Humber's "Handy book of Strains," a little volume which I look upon as indispensable to all having to do with girder work. There you will find much useful information, but it is so condensed that you would make nothing of it unless you had some previous general knowledge, such as I have now given you.

From whatever source you obtain your mode of working, be careful to see that you have the units of measurement correct, because sometimes w stands for the whole load, and

at other times for the load per foot run; in some formulæ s stands for span in feet and in others for span in inches, and so on. In all the cases we have taken, I have described the value of w, and for span we have assumed the same unit as for depth, both being either in feet or in inches.

As regards the nature of the load, we have assumed dead, fixed, or slowly applied loads, but there are cases in which live, moving, or suddenly applied loads must be estimated for, and with these you will obtain different results. The actual strain caused by a moving load is very difficult to determine, and depends partly upon the amount of deflection caused in the structure. This again is subject to the quality of the workmanship, the nature of the material, and the mode of connecting the parts, in such a way that it can only be approximately determined beforehand. From various experiments and theoretical deductions certain allowances are made, which principally affect cast iron, as it is considered to be to a great extent a treacherous material, and therefore a greater margin for safety must be provided. The only allowance of the kind that I need call your attention to now, is in connection with cast-iron girders for bridges, my formula for which is as follows, assuming the ultimate tensile stress of cast iron to be $7\frac{1}{2}$ tons per square inch:

$$\text{s. a. bott. flange} = \frac{\text{span in ft.} \times \left\{ \begin{array}{l} \text{5 times dist. dead load tons} \\ + \text{8 times dist. live load tons} \end{array} \right.}{\text{5 times depth in inches.}}$$

From this you will see that the moving load is assumed to distress the material more than an equal dead load, in the proportion of 8 to 5. The Board of Trade allowance is, I think, rather less than this, but I consider the above a good safe formula and worth noting.

It may be useful to give you a very handy little formula for the mental calculation of wrought and cast-iron girders, per-

fectly accurate under certain conditions, but approximately correct under any other conditions.

Wrought iron dist. load, any span, any load, depth $= \frac{1}{12}$ span
s. a. top flange $= \cdot 3$ W.

Cast iron dist. load, any span, any load, depth $= \frac{1}{12}$ span
s. a. bottom flange $=$ W.

W being total load in tons, and the area in square inches.

In a solid girder, such as a wooden beam or the test bar used for judging the quality of cast iron, the transverse strains are not found in the same way as those in a flanged girder; but as these calculations are not much required by the engineer, and we have previously investigated them in the course of lectures on "Timber," we need not go into it again now.

Angle and tee irons are of course used very largely in positions in which they are subject to transverse strain; the proportioning of the amount of material depends upon a calculation in which the centre of gravity of the section and the neutral axis have to be found. This belongs to designing, but there is something more to say under the head of Strains bearing upon this part of the subject.

When you have an inclined rafter of angle or tee iron, say with a uniformly distributed load, there are two ways of obtaining the value of the cross strain.

Let A B (Fig. 82) be the rafter, and C B the height of B above A, then the calculation may be made the same as for a girder under a distributed load, taking A C as the length or span instead of A B, and bearing in mind that the depth *d* in the formula will depend upon the form of the cross section. Or we may take it another way; and say that $\frac{1}{4}$ the load is carried direct upon each abutment, and $\frac{1}{4}$ transmitted to each abutment *through* the rafter, these $\frac{2}{4}$ acting in the centre and producing a cross strain, the *amount*

of which will vary according to the angle of the rafter, and the *effect* according to the form of the cross section. Rafters are always made of uniform section, and we therefore want only the maximum strain, which will be in the centre. Take $\frac{2}{4}$ths or $\frac{1}{2}$ the load as acting vertically on the centre with the span A C; or by parallelogram of forces, let $d\ e = \frac{1}{2}$ W (Fig. 83), draw $e\ f$ parallel to B A, and $d\ f$ parallel to the direction in which the effect is required, viz., perpendicular to B A, then $f\ d$ will equal the cross strain in the centre upon the length A B and perpendicular to it. This will be a strain for which additional material must be provided beyond that required for resisting the compression transmitted by the rafter due to the arrangement of the trussing.

When a rafter or strut is inclined, although it may have no external load except one acting in the same direction as itself and producing compression, it is still subject to a cross strain from its own weight taken as a distributed load. The ties of a structure are also subject to the same conditions, but being generally lighter, and tensile stress resisting rather than assisting the bending, the effect is not so important.

Vertical struts, as stanchions and columns, are subject to a bending action owing to the practical conditions of manufacture, erection, and loading never being theoretically perfect. So far as the calculation of strains is concerned, all we can obtain is, the dead load producing compression; but in designing columns we have to make allowance for this bending tendency, and, unless tables are referred to, it can only be done by logarithms, which is a hard name for an easy subject.

Joints and fastenings must always be at least equally strong with the parts they connect; as a general rule they are formed of rivets, pins, or bolts, the strain upon them being a shearing strain, tending to cut them through either *at one point* or *at two points*. This question of single or

double shear must be practically noted, because, often you may find that a certain set of rivets &c. can only fail by being sheared through two places at the same time, and then you may take half the shearing force to obtain the requisite sectional area.

We have now taken all the simple cases, forming, as it were, the foundation upon which we can consider the complex cases. The Warren girder forms a very suitable example upon which to try our skill, and we shall proceed with this in the next lecture.

QUESTIONS FOR HOME-WORK.

1. In a girder 20 feet span, 1 foot 8 inches deep, with a uniformly distributed load of 5 tons, including weight of girder, what will be the nature and amount of the various strains at a point 5 feet from the right-hand support ?—Ans:

Compression, top flange	5·625 tons.
Tension, bottom flange	5·625 „
Shearing web	1·25 „

2. A plate girder bridge is continuous over three spans of 15, 20, and 15 feet respectively, its depth is 2 feet, and the load distributed $1\frac{1}{2}$ tons per foot run on each girder; show by diagram the strains throughout. Scales $\frac{1}{4}$ inch = 1 foot and 1 ton.—Ans. (See Fig. 84.) E F = $21\frac{3}{32}$, B G = $28\frac{1}{8}$, I J = $37\frac{1}{2}$.

3. A rafter carries a distributed load of 2 tons, it is inclined at an angle of 30°, what will be the effect in transverse strain on the rafter? —Ans. ·87 tons in centre.

4. What size should the bottom flange of a cast-iron bridge girder be in the centre, when the span is 20 feet, the depth 2 feet, the dead load 5 cwt. per foot run, and the moving load 30 cwt. per foot run?— Ans. 44·17 square inches: say, 18 inches × $2\frac{1}{2}$ inches = 45 square inches.

LECTURE VI.

Open web girders—Warren, lattice, and trellis girders—Angle of inclination of bars—Proportion of depth to length—Warren girder under central concentrated load—Application of parallelogram of forces—All bars equally strained—Strain in flanges cumulative towards the centre of span—Value of strain at each part in terms of x—Vertical ends to girders, how affected by position of last bar—Warren girder under distributed load—Apportioning the load on each span—Action of load from point of application to each support—Bars not equally strained—Strains worked out by parallelogram of forces, commencing at support—Strains on cranes—Ordinary mode of working incorrect—Application of load is diagonally through sheave pin—Angle of jib affects strains—Examples.

We will now apply our knowledge of the transmission of forces to ascertain the strains brought upon the several parts of a girder formed of open triangles, called a Warren girder, as in Fig. 85. When the bars cross each other, as in Fig. 86, it is called a lattice girder, and when several bars cross, as in Fig. 87, it is called a trellis girder.

In a Warren girder the lattice bars are placed at an angle of 60°, making the triangles equal sided, and the bars therefore equal in length to the width of a bay. The *depth* of the girder will always be a fixed proportion depending upon the number of bays in the span, and being $\cdot 866$ or nearly $\frac{9}{10}$ths the width of a bay, this being the proportion the perpendicular height of an equilateral triangle bears to its base.

You will find it handy to note down this sketch of an equilateral triangle (Fig. 88), and the following equations:

$ab = 1$, then $cd = \sqrt{\cdot 75} = \cdot 866$, and $ad = \cdot 5$.
$ab = 2$, „ $cd = \sqrt{3} = 1\cdot 732$, „ $ad = 1\cdot$.
$cd = 1$, „ $ac = 1\cdot 155$, and $ad = \cdot 577$.
$cd = \cdot 5$, „ $ac = \cdot 577$, „ $ad = \cdot 288$.

$$\sqrt{2} = 1\cdot 414 \quad \sqrt{3} = 1\cdot 732.$$

Also the sketch of a right-angled triangle with two equal acute angles, the longest side or hypothenuse forming an angle of 45° with the other two sides (Fig. 89).

$$b\,c = a\,c.$$

$$b\,c = 1,\ \text{then}\ a\,b = \sqrt{2} = 1{\cdot}414.$$

$$a\,b = 1,\ \ ,,\ \ b\,c = \sqrt{\cdot 5} = {\cdot}707.$$

These figures will enable you to sketch down the strains in lattice girders, by the parallelogram of forces, and follow the value of the lines through, without measuring. They are particularly useful if you have no knowledge of trigonometry.

To return to our Warren girder: if the depth appear too great, or too small, the number of bays must be modified accordingly. We will assume only 3 bays in order to reduce our work, but generally you would not have less than 5, and might have as many as 13.

We will consider this girder under four different conditions: with the load concentrated and distributed, and with the number of bays even and uneven.

First, with a concentrated load in the centre on the top flange, and an even number of bays, as in Fig. 90.

We have this load supported primarily by the two struts meeting underneath it, and as we know that half the load passes through each of them towards the support, these will be in compression, and the amount will be found by parallelogram of forces, as in Fig. 91, the vertical dotted line being equal to W; taking one of these struts, we find at the bottom it produces (Fig. 92) tensile strain in the next inclined member, showing it to be a tie, and also in the bottom flange. The other strut produces similar strains on the other side. Then the tie, Fig. 93, brings a compressive strain on the next bar and on the top flange. This bar being the last, the strain transmitted by it is resolved into downward thrust, or dead weight on the support and tension in the bottom flange, as in Fig. 94. The ends of the top

flange, and the vertical ends of the girder do not receive any strain; and the proof is that we have now traced the action of the load right through to the supports without passing through these portions.

Upon putting this down to scale, you will find that with a single central load all the bars are equally strained, and with only three bays the top flange is uniformly strained throughout the two central bars, but the bottom flange has the strain from the *end* struts transmitted through it from end to end, and in the central portion it has the additional strain from the middle struts. Whatever the number of bays, the rule with a central load will be that the lattice bars are all equally strained, but the flanges will be strained in an increasing ratio towards the centre of the span, depending upon the number of bays upon each side producing a cumulative strain.

Now, by reason of our knowing the proportions of an equilateral triangle, we can find the strains very easily by calculation. Suppose our girder to be 15 feet span, the width of each bay will be $\frac{15}{3} = 5$ feet, and the depth of the girder $5 \times \cdot 866 = 4 \cdot 33$ feet, or say, 4 feet 4 inches. Let the load be twelve tons, then the strains will be found thus:

s = span in feet.
n = number of bays.
b = width of bay $= \frac{s}{n}$.
d = depth of girder $= \cdot 866 \frac{s}{n}$.
l = length of bar in feet $= \frac{s}{n}$.
W = concentrated load in tons.
x = strain on centre bars;

then,

$$d : l : \tfrac{1}{2}\text{W} : x,$$

$$\text{or } x = \frac{\frac{1}{2}\text{W}l}{d}, \text{ and this is } = \frac{\text{W}}{1 \cdot 732}, \text{ or } \cdot 577\,\text{W}.$$

In this case, x will $= 6 \cdot 95$ tons, and the strains will be as marked in Fig. 95, the dotted parts having no strain in them.

It might be supposed that the strains in the flanges could be found from the formulæ we have previously taken, or by leverage, and this would be right for the bottom flange of this girder, but not for the top. The strain in the top flange is not due to the direct action of the load, but to the compression resulting from the two bars shown in tension, and the amount would be the same as if we took one bay for leverage, instead of $1\frac{1}{2}$ as we should at the bottom. It will also be noticed that between the points of attachment the various portions of the girder are considered to be under uniform strain, so that a curve of strain say for the bottom flange would be something like a shearing diagram, Fig. 96.

Now we will take the load in the centre again, but use a girder having four bays, the additional bay being placed half at each end of our last girder, as in Fig. 97, increasing the length to 20 feet, the other dimensions remaining the same.

If I thicken the lines in compression, and mark on each part the value of the strains in terms of x, I think you will understand it without going through the working again. (See Fig. 98.)

The dotted portion at each extremity has theoretically no strain, but is of course required to hold the vertical end in its place, and to give sufficient bearing surface for the end of the girder on the supports. It also shows you that a girder thus constructed may be supported at the ends of the top flange, if properly designed, the supports taking the place of the vertical ends.

We will now go back to our first girder of three bays, and put a distributed load of the same amount upon it. From a mere inspection of the bracing we can predict what the nature of the strains will be, and I may therefore just as well draw the girder at once in thick and thin lines, Fig. 99.

If the total weight be 12 tons distributed, and the girder 15 feet long, it will be $\frac{12}{15} = \cdot 8$ tons per foot run. We must see how much of this will come on each apex or supporting point, Fig. 100; from a to b we have $2 \cdot 5$ feet, which $\times \cdot 8 = 2$ tons; from b to c 5 feet $\times \cdot 8 = 4$ tons, c to d again 4 tons, and d to e the same as the other end $= 2$ tons. Now, of the first 2 tons half is borne by the vertical bar at the end and the other half by the first pair of inclined bars. Then, with the 4 tons, we have half on each apex at the sides of that bay, and so on. Adding these half-loads together, we get 1, 3, 4, 3, 1.

In order to get the strains correctly with a distributed load, or a load or loads out of the centre, it is necessary to work from the abutments towards the centre, and we must therefore find the pressure on the abutments next. To do this, we find the centre of gravity of the load, and obtain the result by leverage as previously explained. In this case, as the load is symmetrical, the pressure on each abutment will be half the total load, or 6 tons. But there is 1 ton acting exactly opposite to it in e B and a A, so that the amount acting through the girder trussing will be $6 - 1 = 5$ tons. Tension and compression may be conveniently indicated by the signs $-$ and $+$. By parallelogram of forces, Fig. 101, we resolve the 5 tons at B into $2 \cdot 9$ tons in g B, and $5 \cdot 8$ tons in d B. Transferring the strain in d B to its upper end, we then have the load on d, and the strain in d B, both acting at point d, and we want to find the effect in $g\,d$ and $c\,d$. Before this can be done, we must compound the two forces $+ 5 \cdot 8$ and $3 \cdot 0$ into one, as shown at $h\,d$, then resolve its effect into the two parts $c\,d$ and $g\,d = 4 \cdot 0$

and 2·3 respectively. Transferring the strain in $d\,g$ to its lower end, we have it acting upon $f\,g$ and $c\,g$, producing strains in each, equal to 2·3. Now we add the strain in Bg to that in $f\,g$, and we obtain the accumulated strain in $f\,g$ = 5·2. The other half of the girder will be exactly similar.

I worked the strains in the first example this evening from the load towards the abutment, because it appears to be more direct, and when we omit the weight of girder there is no difficulty; but you must take it as a general rule that in any case the strains should be worked from one abutment towards the other. In practice, of course, you never get a concentrated load alone: you have always the weight of the girder itself as a distributed load in addition to the other. The weight of girder is allowed by taking it as being divided over the various points of support, viz. the apex of each bay, precisely in the same way as you would consider it if it were an *external* distributed load.

I have now shown you sufficient of the application of the parallelogram of forces to lattice girders to enable you, with time and patience, to work out any example that may arise; the consideration of other arrangements of trussed girders will be better left until after the next lecture, as I shall then introduce to you a system of working by which your difficulties will so vanish that it may well be called the royal road to the determination of strains.

There are, however, a few cases of structures in which the strains are determined more easily by the parallelogram of forces than by any other method. Among these are all varieties of cranes, and as I have not yet seen any book which indicates the mode of finding the strains correctly, it may be desirable to give you a few hints about them.

In its simplest form, a crane is a projecting bracket, carrying a pulley, over which a rope or chain passes. If a crane of the kind shown in Fig. 102 have a load of 1 ton, the books say the strain will be tension = 1 ton, compression

$= \sqrt{2} = 1 \cdot 4$ tons, as in Fig. 103; but if we take this sketch as a sample, and apply the method without discrimination, we shall soon get into difficulty. In the first place, the load is not hanging direct from the end of the jib; it is hanging on the chain, and the chain has an equal tension in every part, and we have therefore two forces, as in Fig. 104, the resultant of which will pass through the sheave-pin, and give us a diagonal force as the actual connecting link between the load and the crane. With the chain leading off horizontally from the top of the sheave, and the load always hanging vertically, we shall find the effect the same as if the load were hanging direct from the sheave-pin, so far no error; but frequently the chain is not led horizontally from the top, and allowance should be made in many cases for pulling at the load with the chain out of plumb. So that practically we must take the direction of the chain for the top of the parallelogram, and the mean direction of the load for the side of the parallelogram. Then the remainder of the working will be a simple application of the principle that you have already had some practice in.

If we had taken an angle of 60° for the jib, the error of the common mode of working would have been very striking. Fig. 105 shows the arrangement, and enlarging the parallelogram of forces, as in Fig. 106, we find that the tension in the stays will be given as ·58 ton, and compression in jib 1·16 tons; whereas, taking the correct mode as in Fig. 107, or enlarged in Fig. 108, we find that both the jib and mainstay or tie rod are in compression, the amounts being respectively 1·16 tons and ·42 ton. The weight of the parts will affect the precise result, but not the principle. This shows how very careful one must be to take the exact course by which the strain is communicated from one part to another.

The subject of Cranes will form one of the lectures in the Advanced Course, which I propose to give as a sequel to the

present course; there is ample work for an evening in tracing out the various strains upon a few of the more typical forms, and I must not detain you longer with them now.

QUESTIONS FOR HOME-WORK.

1. Mark the amount of strain in each part of the following truss (Fig. 109), when the load equals 3 tons, omitting weight of truss.—Ans. See Fig. 110.

2. In the following girder (Fig. 111) mark the strains in each portion, taking the distributed load as 10 tons, including weight of girder.—Ans. See Fig. 112.

3. In the following girder (Fig. 113) distinguish the parts in compression by thick lines.—Ans. See Fig. 114.

LECTURE VII.

The reciprocal diagram of forces—Explanation of the principle—General knowledge of strains necessary before use can be made of this system—Construction of diagram—Lettering spaces instead of lines—General directions for constructing diagrams—Application to roofs—Diagrams for single and double rafters—For span roof with tie rod—For same with addition of king bolt—For ordinary king-post roof truss—For roof with king and queen rods—For span roof with tie rod and double king bolt—For same with addition of struts—For same having additional ties and struts for larger span—For span roof with king bolt and cambered tie rod—For same with open king bolts—For trussed rafter roof—Examples.

We have seen how to find the strains in the simplest form of a roof truss, or trussed beam, by means of the parallelogram of forces, and you will remember that I mentioned the fact, that if any three forces in equilibrium acting on a point be represented by three lines of proportionate magnitude and corresponding direction, they could be so arranged as to form a perfect triangle; and any number of forces, under similar conditions, would form a closed polygon of the same number of sides.

Well, the converse or reciprocal of that statement holds good, and upon this is founded the system first introduced by Professor Clerk Maxwell, called the "Reciprocal diagram of forces"; which has been further elaborated by Mr. Bow, a civil engineer, and I have now to introduce it to you with a few applications of my own.

Briefly, the reciprocal relationship would be defined thus: corresponding lines which meet in a point in the one figure form a closed polygon in the other, but this is not always possible so far as regards lines representing external forces.

We will take a simple triangular truss to represent the method, and then you will better understand the directions which I shall give you for finding the reciprocal diagram in any given case.

Let Fig. 115 be the truss supported at each end, and, for sake of simplicity, loaded with a concentrated weight at the apex.

Now, in this system it is of vital importance to ascertain correctly the position and amount of all external forces; here we have three, I will indicate them by arrows (see Fig. 116). The next step is to letter each space within the trussing, and also each space outside the trussing, as in the figure.

Then, in the diagram, any line representing a portion of the truss, or a line of force external to it, will be named from the two letters belonging to the two spaces it separates.

Thus, draw C D (Fig. 117), in direction and magnitude equal to the load on apex; it is evident that the reaction on each support will equal $\frac{1}{2}$ C D, or C B + B D = C D, therefore place B midway between C and D. From C draw C A parallel to the left-hand rafter, and B A parallel to tie rod, meeting in point A; also D A parallel to right-hand rafter, when the figure is completed; and by measuring the length of any line in the diagram we obtain the strain acting in or upon the part separating the similarly lettered spaces in the truss. To make the diagram indicate as much as possible, the parts in compression should be thickened, and you will have no difficulty in doing this with your present knowledge, although the diagram throws no light upon this portion of the subject.

You will now be in a position to understand the detailed directions for constructing a reciprocal diagram for any other case.

1st. Assign a letter to each enclosed area of the truss, also to each division of the surrounding space as separated by the lines of action of the external forces.

2nd. The lines in the diagram are to be drawn parallel to the corresponding lines or parts of the truss figure.

3rd. The forces acting in lines radiating from a point in the truss must in the reciprocal diagram form a closed polygon.

4th. The sides of any uncrossed space, triangle, or other polygon, around or in the truss, must always be represented by lines radiating from a point in the reciprocal diagram, and that point is to be named by the letter assigned to the space or polygon.

5th. All the external forces acting upon a truss must, taken together, be represented in the reciprocal diagram by the sides of a closed polygon.

In the example we have taken above, the external forces, taken together, really form a closed polygon, but the sides overlie each other. If we open them a little, as in Fig. 118, we shall see this relationship better.

Although the principles are so simple, the arrangement of the diagram will sometimes be a troublesome matter until you have had considerable experience, and I therefore propose to give you sketches of some of the ordinary forms of roof trussing in ironwork, with the corresponding diagram fully lettered, and in fact ready for you to apply to any case that may arise in your practice.

We will start with our former illustration of the inclined beam, and divide the trussed examples into groups, according to the arrangement of the parts, assuming in every case a distributed load such as a roof has ordinarily to bear.

For a single rafter, as in Fig. 119, commence with the loads, taking the left-hand first, and if your sketch has been lettered properly and the diagram worked correctly, the letters on the diagram will come in regular order, as shown in Fig. 120.

For a pair of rafters we have the arrangement shown in

Fig. 121, with its diagram shown in Fig. 122,* and for the same with a tie rod we have Figs. 123 and 124. In this last diagram we have a clear illustration of what I have told you previously about the portion of the distributed load next the abutment being carried direct by the support, without transmitting any strain through the truss. Here you see the piece C D comes outside the lines showing the strains in the truss, indicating that it is dead load on the abutment only.

Now, if we put a king bolt in the same truss, as in Fig. 125, we shall find in Fig. 126 that it has no strain from the load, its sole duty being to support the tie rod. C A, C B are both horizontal, and must therefore be in the same straight line from C; but E A and F B are equal in length, start at equal distances from C, and make equal angles with E F, therefore A and B must be both at the same point; then the strain in A B being represented by the distance from A to B, it will be seen to be nil. Many other cases will occur in practice, in which you will find that, so far as the transmission of the load is concerned, there will be a redundancy of parts indicated by the letters, representing a line, meeting in one point. In other cases you may find that there will be an excess of parts giving a variable strain according as the load is assumed to be borne more or less by the different members; these latter are of course awkward instances, and belong to advanced practice.

Now we will add two struts to the truss, and try the result. (See Figs. 127 and 128.) In this case A and D come into one point, because the tie rod is straight; if it were cambered they would not come together.

Taking the same type of roof applied to a larger span, as

* In the lectures the whole of the illustrations were drawn on the blackboard, the diagrams being constructed line by line; it is hoped that the reader will find no great difficulty in tracing out for himself the order of working and the lengths of the lines with the lettering given.

in Fig. 129, we find, as we might have expected, that there is no strain in I J, or A B. (See Fig. 130.)

Now go back to Fig. 125 and split the king bolt so as to get two points of support for the tie rod, as in Fig. 131, and see what effect it has. We find by Fig. 132 that there is no strain in A B or B C due to the trussing, so that the only strain upon them will be due to the weight of the tie rod.

Let us add two struts, and we get the truss and diagram shown in Figs. 133 and 134.

Taking the same type of roof for a larger span, the additional trussing will be as shown in Fig. 135, and the diagram as in Fig. 136. In this form of roof the struts are sometimes arranged so as to be perpendicular to the rafters, and therefore in the best position for resisting the strain in them, because they are then of the minimum length. At other times they are arranged by dividing the tie rod equally and making the junctions with the tie rod at the points so found.

Next go back to Fig. 125 and camber the tie rod, as in Fig. 137; owing to the camber of the tie rod, we now get a strain in A B, as seen in Fig. 138.

Now follow on with Fig. 131 cambering the tie rod in the same way, as shown in Fig. 139, the diagram for which will be Fig. 140.

Take the same again with the addition of struts, as in Fig. 141, giving a very common form, and known as a trussed rafter roof. We may consider this as two separate trusses, united by a tie, as in Fig. 143: it is a very good form for small roofs.

In this system of diagrams you have a key to the elucidation of the strains in the most complicated structures, and I shall further exemplify its use in the next lecture by taking various forms of lattice girders.

Very careful working is necessary, and accurate parallelism between the lines of the diagram and those of the figure.

The examples for home-work upon the present lecture will form a very good test, both of your carefulness in this respect and of your general ability in applying known rules to modified cases.

QUESTIONS FOR HOME-WORK.

Work out the strains in the following trusses by means of reciprocal diagrams, and figure the values on the sketches:

1. Span 20 feet, load 4 tons distributed, camber of tie-rod 12 inches, rise $\frac{1}{4}$th that of span. See Fig. 144.—Ans. See Figs. 145 and 146.
2. Span 20 feet, load 4 tons distributed, camber 12 inches, rise $\frac{1}{4}$th of span. See Fig. 147.—Ans. See Figs. 148 and 149.

LECTURE VIII.

Application of reciprocal diagram to lattice girders—Diagram for Warren girder, with central load supported on top flange—For same with load on bottom flange—For same with additional bays—For same with distributed load on top flange—For same with load on bottom flange—For lattice girder, with bars intersecting at 45° under concentrated and distributed loads—For lattice girder with vertical bars—Examples.

This evening we shall apply the reciprocal diagram of forces to lattice girders of different kinds, and you will see how favourably the system compares with the ordinary parallelogram of forces for such investigations.

Very little explanation will be required; the chief difficulty is to know what form the diagram will take for each variety of girder or arrangement of bars. I will sketch them down, and point out any particular part requiring notice, and I hope you will stop me at once, if any part is not quite clear to you, in order that I may explain it.

We will take first a Warren girder of 3 bays, with a concentrated load in the centre on top flange, Fig. 150. In this diagram, Fig. 151, we have I J = full load, and I H, H J each = $\frac{1}{2}$ full load. There is no strain in I A or J G, which agrees with the result found by other methods.

But suppose we have the same load supported by the bottom flange instead of the top, as in Fig. 152. We now get considerable alteration in the strains, and find some difficulty in adjusting the lines showing the application of the external force or load. The weight is acting in the centre of middle

bay, but so far as the trussing is concerned, the load is divided between the points *a* and *b*, and we must therefore put lines here, each representing half the load, and then letter the spaces as shown. The letter K represents the whole exterior of the girder above the abutments, because there is no external force outside of the reactions upon the abutments. Then, in constructing the diagram Fig. 153, we shall find that as the load is on the bottom flange the whole diagram will be reversed, the truss strains being on the right of the lines of forces instead of on the left, and you see that C D, D E have no strain upon them, because all three letters meet in one point in the diagram.

Now take the first example again, and add half a bay on to each end, making 4 bays in all instead of 3, as in Fig. 154, giving the diagram in Fig. 155.

If the load were taken on the bottom flange of this girder, you would work it out in the same way as Fig. 152, but the diagram would be like Fig. 151 reversed.

Now take Fig. 150 again, and try it with a distributed load, Fig. 156, obtaining the diagram Fig. 157.

If you compare diagrams 151 and 157, you will see that with the same total load the strains are much smaller with the load distributed, shown by the area of the diagram being smaller when drawn to the same scale.

We will take the same girder again with a distributed load, but carried by the bottom flange, because we shall then discover a new difficulty. (See Fig. 158.) The difficulty is this: at each end of the girder next the abutment we want a line to indicate the portion of the distributed load carried by that point, and also a line indicating the reaction of the abutment, both lines being truly vertical, and meeting at the same point. In a case like this, the simplest way is to imagine the double line split open, giving room to put a letter between, as shown at L and H. Then I H will be the

load and H M the reaction, and the diagram will be as Fig. 159.

Next we will take a lattice girder, the bars of which cross each other in the centre of their length, and all incline at an angle of 45°, as in Fig. 160. It is very certain that, without being told, many of you would puzzle over this girder a long time before you would be able to satisfy yourselves about the strains, and you will most likely be astonished to see the number of theoretically useless parts, as proved by Fig. 161. Practically, the dotted bars would help to stiffen the others, and therefore be of some service, a bolt or rivet being put through their intersection.

Taking the same girder with a distributed load on the top flange, as in Fig. 162, we shall find by diagram Fig. 163, that all the bars but two are strained. Point A in the diagram will be midway between P and Q.

Suppose we add vertical struts to this girder at each apex, giving Fig. 164, we shall then find by Fig. 165 that even with a concentrated load all the bars are under strain except two struts which are not underneath the load.

For the next example we will take the same girder, but with a distributed load carried by the *bottom* flange, Fig. 166. In this case, taking the letters in alphabetical order, W happens to come on top, where it represents the exterior space, not a load, and in setting off the loads in the diagram Fig. 167, we have some difficulty in finding point A. It is obtained thus: W V is half the load, from this subtract V U, which is the amount borne direct by the abutment; the remainder U W will be transmitted half through A B and half through A C, therefore place A midway between W and U. In this case we have a perfect girder, every part being under strain.

In applying reciprocal diagrams to the study of strains in bridges, great caution is required, owing to the magnitude

of the expenditure and credit at stake, and the difficulty in many cases of deciding the exact course of the strains. While the method is second to none for estimating the proportions in preliminary designs, precise calculations should be resorted to for the final sections and joints.

Many other structural forms might have claimed our notice, but as our time would only permit of examining a few leading varieties which could be treated in an elementary manner, I have selected those which I thought would be most useful to you, and I hope that you will be able to make practical use of the information you have derived here.

In conclusion, allow me to thank you most heartily for the interest you have shown in this comparatively dry subject, and particularly for the manner in which you have worked the home papers; although there have been numerous failures, there has on the whole been substantial progress. I always adopt the custom of giving questions for home-work, because it compels some attention to the subject between the lectures, and the answers enable me to gauge the ability of my hearers so that I may modify the future lectures if necessary; and, moreover, the pleasure of personal conference with my hearers is not the least of the advantages. In all probability the scheme inaugurated by the lectures now brought to a close will become a permanent feature of the Society of Engineers, and therefore any suggestions as to future subjects will be gladly received. I trust you will make these courses widely known among your professional friends, and I may say in response to numerous enquiries from those anxious to continue the study of strains, that I will endeavour to get an Advanced Course ready for next session, but my numerous engagements leave me so little leisure, that it may be some months before I can make a commencement. Cranes, gas-holders, lock-gates, bridges, arched roofs, and the effect of moving loads will form some of the subjects; and should any of you

in the meantime discover any structure involving special difficulties, I shall esteem it a favour if you will communicate with me, in order that they may receive due consideration.

QUESTIONS FOR HOME-WORK.

1. A Warren girder, 20 feet span, has a concentrated load of 5 tons, carried as shown in Fig. 168. Indicate the nature of the strains and figure their value, omitting the weight of the girder.—Ans. See Figs. 169 and 170.
2. A lattice girder (as in Fig. 171) for bridge 24 feet span, 6 feet deep, weighs 4 tons, and carries a distributed load of 2 tons per foot run on bottom flange; show the nature of the strains and figure their value.—Ans. See Fig. 172, and for diagram see Fig. 167.

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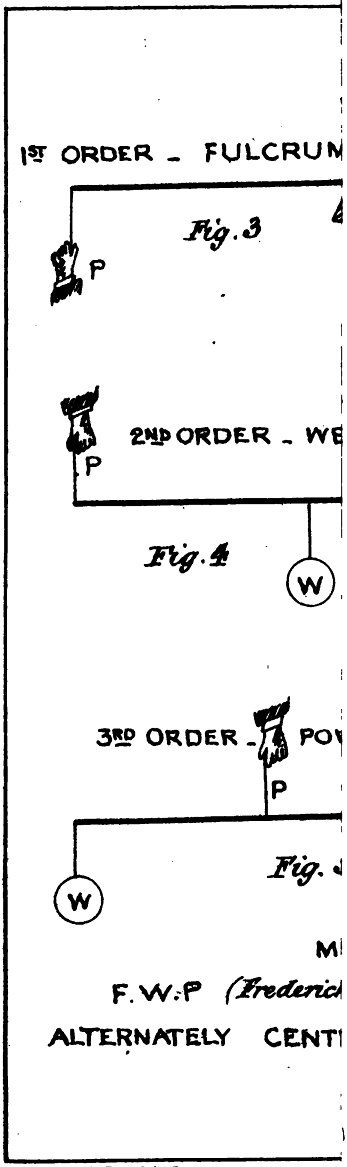

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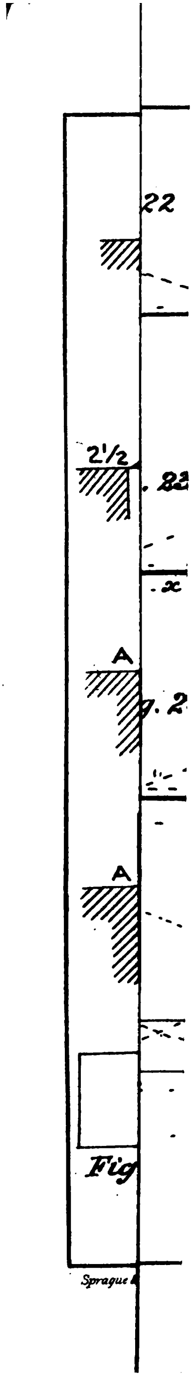
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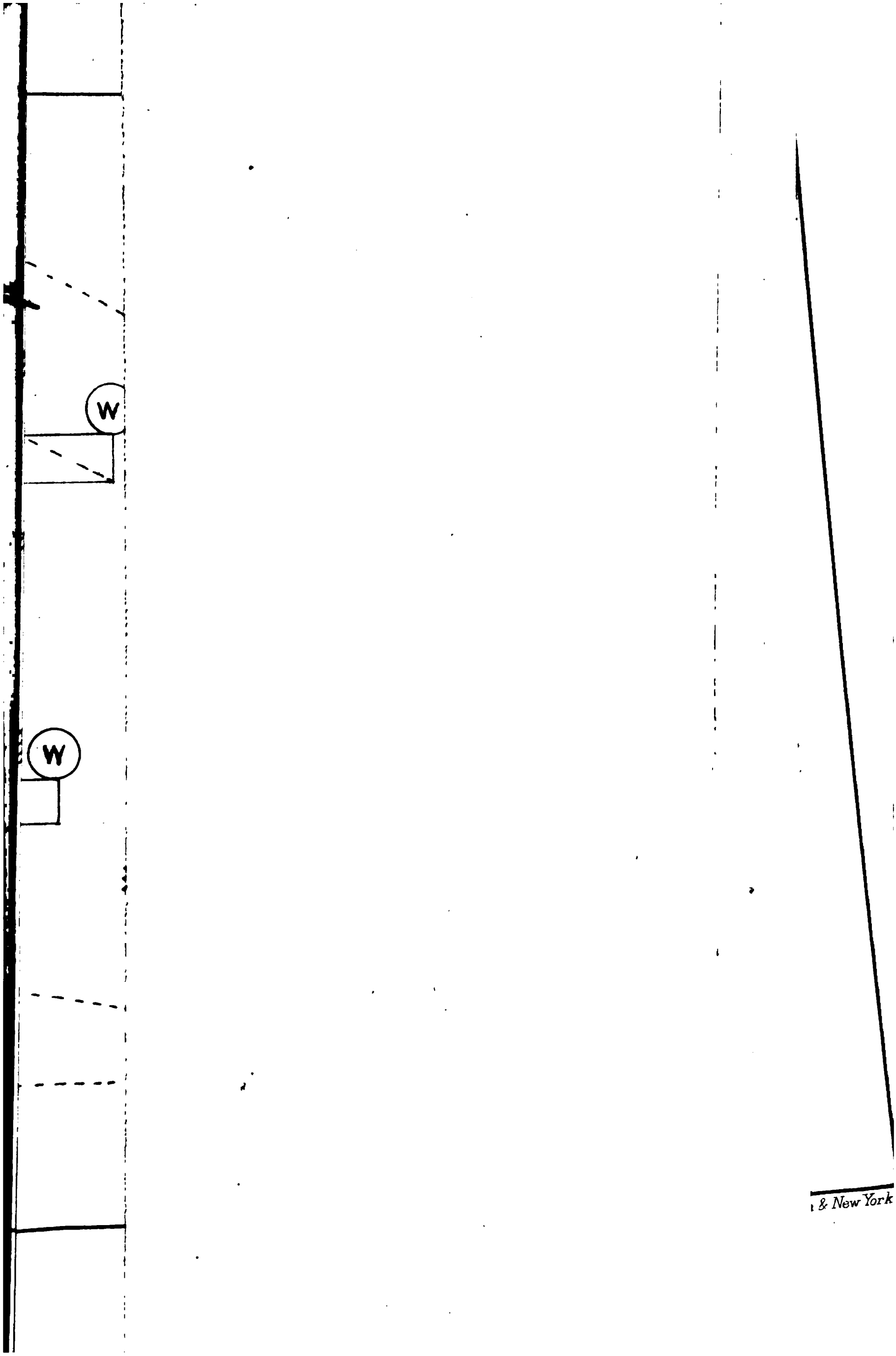
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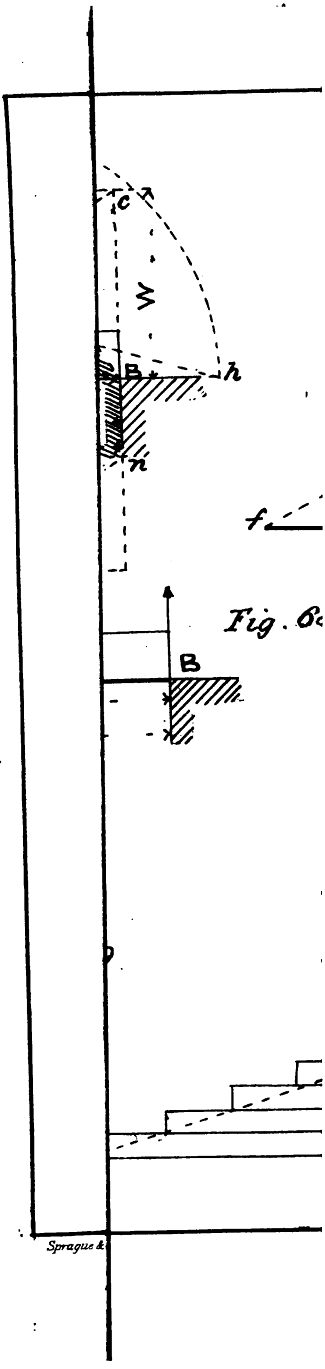

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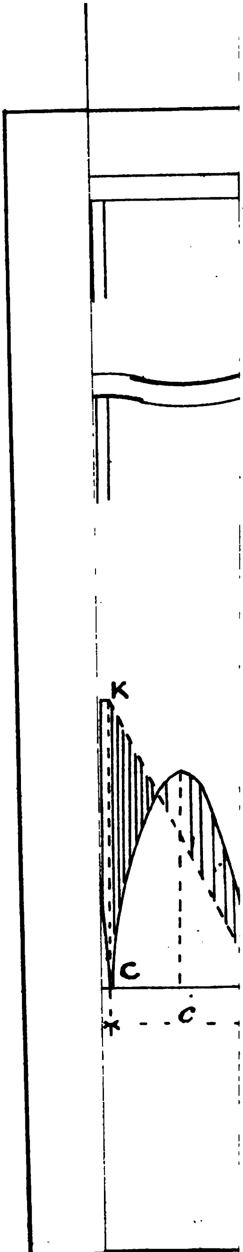

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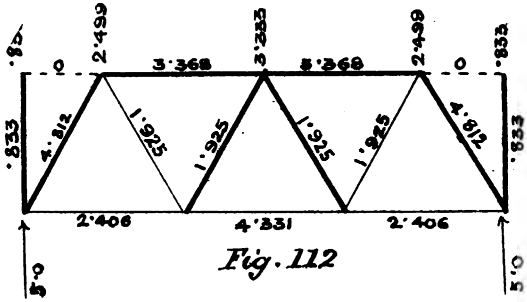

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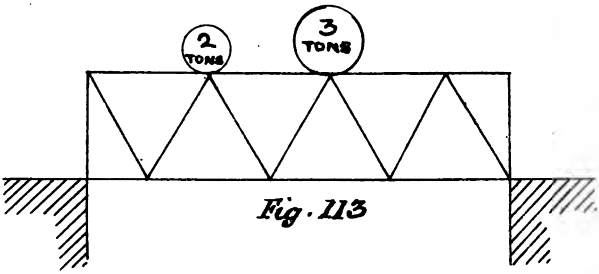

Fig. 113

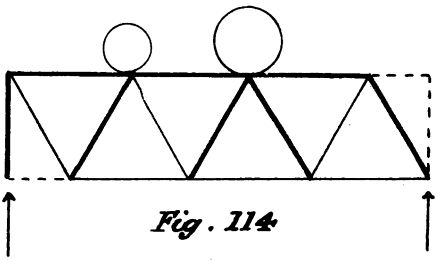

Fig. 114

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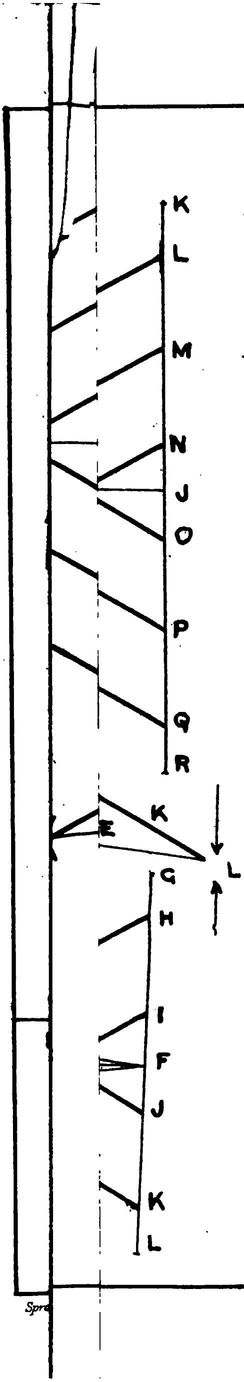


Fig. 144

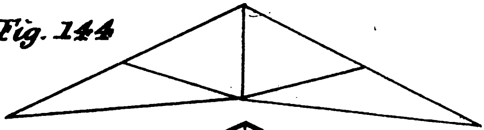
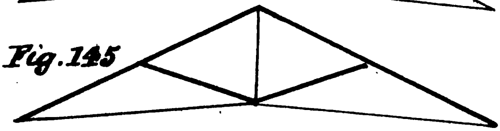


Fig. 145



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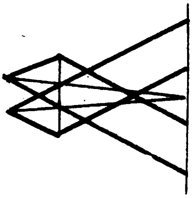


Fig. 147

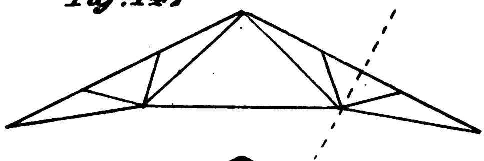


Fig. 148

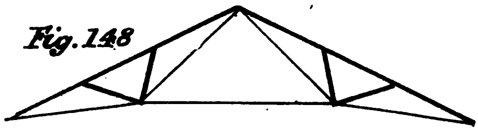
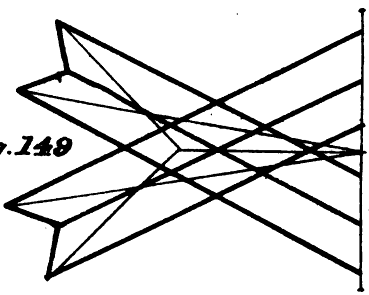


Fig. 149



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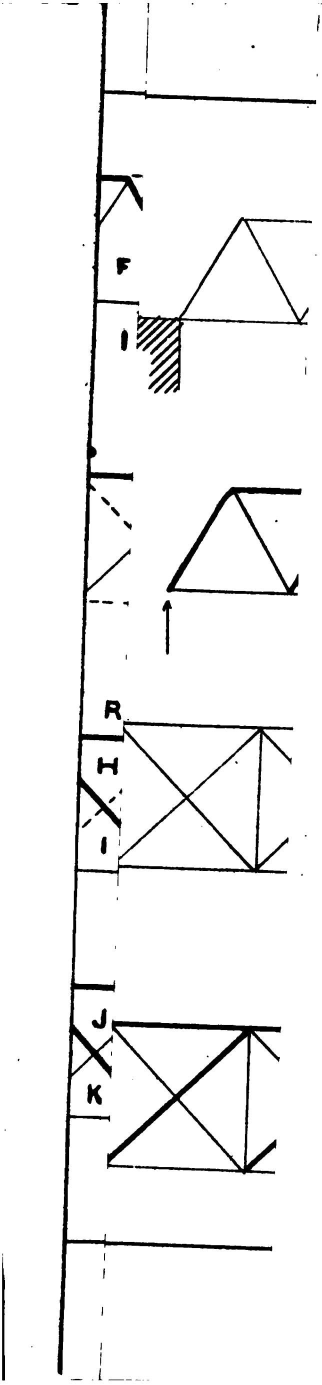
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